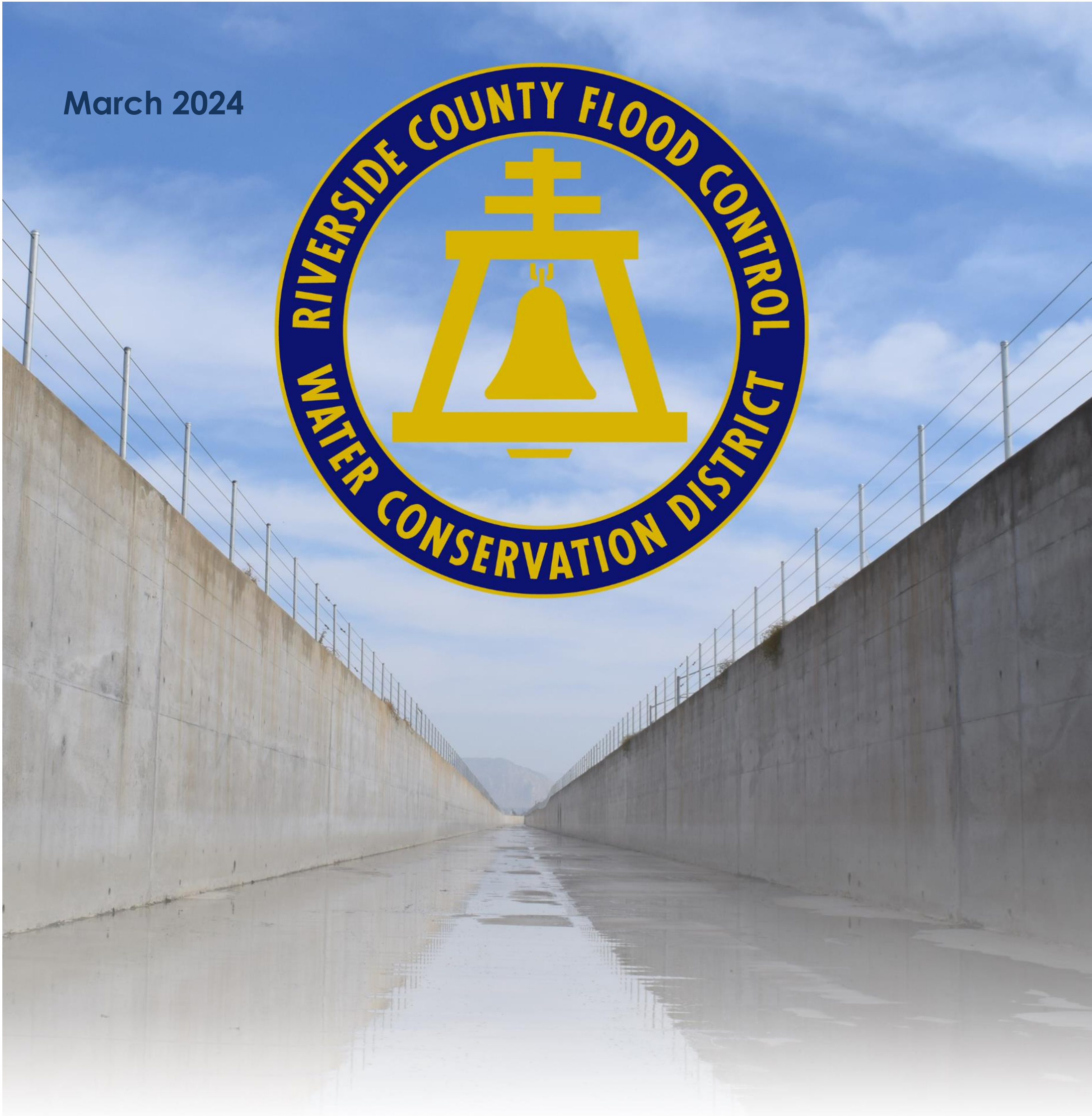
March 2024



Hydraulic Design Manual

for Riverside County Flood Control and Water Conservation District

Jason E. Uhley
General Manager - Chief Engineer

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Acknowledgments

This manual is by the Riverside County Flood Control and Water Conservation District (District). A work group composed of District and CValdo Corporation staff was formed to determine the applicability of technical criteria, special problem areas, and resolve conflicts over potential differences in drainage standards between communities.

The first edition of this manual was released in a draft format (October 2022) for review and comment by public and private sector engineers and other interested parties. District staff revised the manual using comments received from the public and reissued the manual with a date of March 2024.

The Riverside County Flood Control and Water Conservation District wishes to thank the many individuals who contributed to the preparation of this document especially assistance provided by Maricopa County, San Diego County, and Orange County.

Much of the information and recommendations presented in this manual are the result of previously published efforts of many talented engineers and scientists. The District has made every effort to cite the original authors and researchers whose contributions form the basis of the manual.

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The following are acknowledged for their respective work:

- Ava Moussavi for the cover photo of Heacock Channel. Evan de Chambeau for cover graphics.

Comments

Users of this manual are welcomed to submit comments, suggestions, or findings of errors. This information should be addressed to:

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Revisions

Because of ongoing technical and administrative changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as needed basis and will be posted on the District's Web page (www.rcflood.org). The dates of revision and an overview of changes made are listed below.

The District's General Manager-Chief Engineer is hereby authorized to approve future revisions to the Hydraulic Design Manual.

1st Edition – draft	October 2022
2nd Edition	March 2024

APPROVAL BY GENERAL MANAGER-CHIEF ENGINEER

This Hydraulic Design Manual is hereby approved for use when designing facilities for the Riverside County Flood Control and Water Conservation District. This manual has been reviewed by Riverside Flood Control and Water Conservation District (District) staff, other local agencies, engineering consultants, the development community, and the Public at large. Review comments have been addressed within the document.

The latest edition of this Hydraulic Design Manual is only available in digital format and can be found on the District website at:

https://rcflood.org/engineering-tools

Refer to the Revisions Section of the manual for a history of the changes made.

The objective of the Hydraulic Design Manual is to provide criteria and design guidance for storm drainage facilities for the Riverside County Flood Control and Water Conservation District. This manual provides a convenient source of technical information that is specifically tailored to the unique hydrologic, environmental, and social character of Riverside County; and a consistent set of criteria that, when used by the local governing agencies and the land development community, will result in uniform drainage practices throughout the County.

Submissions made using other methodologies may be acceptable to the Riverside County Flood Control and Water Conservation District on a case-by-case basis, upon submission of scientific documentation and evidence showing that such methodology yields results that are consistent with this manual and in accordance with the requirements of the various ordinances and regulations. However, the burden of proof is on the applicant and may affect submittal review times.

Approved for use by:

3-27-2024

Jason E. Uhley, P.E. Date
General Manager-Chief Engineer
Riverside County Flood Control and Water Conservation District

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Abbreviations and 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
CDFW	California Department of Fish and Wildlife
CWA	Clean Water Act
DAMP	Drainage Area Management Plan
DSOD	Division of Safety of Dams
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
MS4	Municipal Separate Storm Sewer System
MUSLE	Modified Universal Soil Loss Equation
RCFCD	Riverside County Flood Control and Water Conservation District (District)
RCP	reinforced concrete pipe
RWQCB	Regional Water Quality Control Board
SPPWC	Standard Plans for Public Works Construction "Greenbook"
SSPWC	Standard Specifications for Public Works Construction
SWRCB	State Water Resources Control Board
USACE	United States Army Corps of Engineers (ACOE)
USBR	United States Bureau of Reclamation
USEPA	United States Environmental Protection Agency

Design Abbreviations

ρ	density of water
τ_0	shear stress
γ_s	specific weight of stone, lb/ft^3
γ_W	specific weight of water, lb/ft^3
a	curb inlet depression, inches
A	area
B	base width of culvert opening, ft
C	rational equation runoff coefficient
C_o	orifice coefficient
Crown	highest point in street
cfs	cubic feet per second
C_W	weir discharge coefficient
d	flow depth, ft
d_C	critical depth
d_{50}	diameter of rock particle for which 50% of the gradation is finer by weight (other percentages may also be used)
D	diameter of pipe in inches
D/S	downstream
E	energy and/or specific energy, ft
EGL	energy grade line
F_r	Froude number
fps	feet per second
g	gravitational acceleration, 32.2 ft/sec^2
h	sum of inlet loss, friction loss, and velocity head in a culvert, ft
h_b	head loss through a bend of a culvert, ft
h_i	head loss at inlet, ft
h_j	head loss through a junction, ft
h_v	velocity head, ft
h_{minor}	minor head loss
HGL	hydraulic grade line
HW	headwater: depth from inlet invert to upstream total energy grade line, ft
k	pipe friction loss coefficient, dependent on Manning's n
k_b	bend loss coefficient
k_c	coefficient for transition loss due to contraction of flow
k_e	coefficient for transition loss due to expansion of flow
k_{en}	entrance loss coefficient

k_i	coefficient for transition loss due to contraction of flow
k_j	junction loss coefficient
k_o	coefficient for transition loss due to expansion of flow
L	length
L_a	length of apron, ft
L_t	curb opening length to intercept 100% of gutter flow, ft
n	Manning's roughness coefficient
P	Perimeter
Q	discharge, cfs
r	radius of curvature at centerline of channel
R	hydraulic radius, ft
R/W or ROW	right of way
S	longitudinal slope
S_f	friction slope, ft/ft
S_X	street pavement cross (transverse) slope
t	time, minutes
T	width of flow, spread on pavement
T_c	time of concentration, min
TW	tailwater depth measured from culvert outlet invert, ft
U/S	upstream
V	velocity, ft/sec
V_a	approach channel velocity, ft/sec
W	width of gutter, ft
W_a	width of apron, ft
WSE	water surface elevation, ft
y	depth of water, ft
y_c	critical depth of flow, ft
y_n	normal depth of flow, ft
y_s	depth of scour, ft

1 INTRODUCTION

1.1 PURPOSE

The objective of the *Hydraulic Design Manual for Riverside County Flood Control and Water Conservation District* is to provide criteria and design guidance for stormwater management facilities in Riverside County. There are two reasons to develop such a manual: 1) it provides a convenient source of technical information that is specifically tailored to the unique hydrologic, environmental, and social character of Riverside County; and 2) it provides a consistent set of criteria that, when used by the local governing agencies and the land development community, will result in more consistent stormwater management practices applied throughout the County. Use of the Hydraulic Design Manual will result in improved hydraulic performance of stormwater management facilities, greater uniformity in design practices across jurisdictional boundaries, provide more predictable outcomes for the land development community, and ensure consistent approaches to design and construction that will facilitate the long-term maintainability of these facilities.

When a facility will be accepted by the District for ownership and maintenance, this manual sets forth design guidance that is acceptable to the District for such facilities.

Each municipality may adopt its own policies and standards. In most cases, these policies and standards are expected to be the same or only slightly modified as presented in this manual. Nevertheless, the user is strongly encouraged to confirm with the local land use authority whether this manual or other standards may apply to each project, and review and account for any differences between policies and standards.

This manual is intended to provide guidance for the design of new construction and major reconstruction of stormwater management and flood hazard mitigation facilities. This manual is not a textbook or a substitute for appropriate engineering knowledge, experience, or judgment. No attempt is made to detail basic engineering principles. The fact that new minimum design standards are presented does not imply that existing facilities are in any way substandard or inadequate. The approaches contained herein will provide more satisfactory design for new facilities and for major modification of existing facilities.

Approval of this manual by the District does not in any way absolve the owner, developer, or design engineer of their respective responsibilities for design. The design engineer has the responsibility to design stormwater management facilities that meet standards of practice for the industry and promote public safety. Meeting the policies and minimum design standards does not constitute a guarantee that properties will meet all required standards, or that the property will be free from flooding or flood damage in all storm events. The District, its officials, employees, and contractors assume no responsibility for any information, data, calculations, results, or conclusions developed by private engineers or environmental professionals and makes no warranty, expressed or implied, in its review/approval of their work products.

This manual limits its content to the planning and design of stormwater conveyance and flood hazard mitigation infrastructure. For issues pertaining to stormwater quality, readers are directed to other resources, and specifically to Water Quality Management Plan Guidance Documents for the appropriate jurisdiction and watershed.

1.2 SCOPE

The *Hydraulic Design Manual for Riverside County* is divided into eleven chapters that address the major subject areas of hydraulic design. The intent of this manual is to provide general design guidance for designs that are common to the Riverside County environment. Complex designs requiring specific expertise are not covered in this manual; however, where the design challenge exceeds the scope of this manual, the user is referred to documentation that is appropriate for such design. The following Sections briefly summarize each of the chapters in the manual.

1.2.1 Introduction

Chapter 1 defines the purpose, background, and scope of the manual along with a brief summary of each chapter. It also includes a discussion of District policies, recreational uses, drainage law, District authority and jurisdiction and supplemental disciplines.

1.2.2 Submittal Requirements

Chapter 2 provides an overview of the submittal requirements for reports, entitlement and plan check submittals to the Riverside County Flood Control and Water Conservation District.

1.2.3 Hydrology

Chapter 3 provides an overview of the hydrology criteria for stormwater management facilities pursuant to the District's Hydrology Manual. The District's Hydrology Manual provides for the use of the Rational Method for small, uniform watersheds; and for use of the Unit Hydrograph Method for larger watersheds with diverse surface conditions. The Hydrology Manual provides design rainfall criteria that have been developed specifically for Riverside County, rainfall loss methods available for estimating surface retention losses and infiltration rates, and unit hydrograph procedures that have been selected and developed for the various land-uses in Riverside County. A discussion regarding the use of bulking factors is included.

1.2.4 Hydraulics of Gravity Flow Drainage Systems

Chapter 4 provides an overview of the hydraulic criteria for stormwater management facilities utilizing gravity flow. Flow classifications, energy states and flow regimes are summarized. Various energy losses to be considered in hydraulic calculations are provided. Example problems are provided utilizing the principles and equations covered in this chapter.

1.2.5 Street Drainage

Chapter 5 provides design guidelines for street drainage using curb inlets and other types of storm drain inlets. An overall approach to stormwater management includes using the street system to transport runoff to storm drain inlets, and for transporting runoff from storms that exceed the capacity of the storm drain system. Design criteria, design procedures, and design aids are provided for streets and gutters, and intersections. Catch basins are discussed regarding alternative types and suggested applications, capacities, and design procedures. The procedures used in this chapter were primarily adapted from the Los Angeles County Flood Control Hydraulic Design Manual and the Federal Highway Administration Hydraulic Engineering Circular No. 22 (HEC-22), *Urban Drainage Design Manual* (FHWA, 2009).

1.2.6 Storm Drains

Chapter 6 covers underground storm drain designs. A comprehensive treatment of storm drain design is provided including use of design aids for catch basins, manholes, and various types of storm drain junctions.

1.2.7 Culverts and Bridges

Chapter 7 covers the design of roadway culverts and bridges. This includes the necessary design aids, guidance for treatment of culvert inlets and outlets, and scour protection at the culvert outlet. Use of example problems helps to illustrate the procedures to be used for most practical applications. The charts and procedures for culvert design used in this chapter were taken from the Federal Highway Administration Hydraulic Design Series No. 5 (HDS-5), *Hydraulic Design of Highway Culverts* (FHWA, 2012). The design of bridges requires special expertise and experience regarding hydraulic analyses, design of flow training works, and estimates of pier and abutment scour. Therefore, only an overview of the hydraulic analyses for bridge openings is presented.

1.2.8 Open Channels

Chapter 8 is devoted to the analysis of both natural watercourses and man-made/engineered open channels. The scope of this chapter covers the more commonly encountered open channel design applications. The design procedure presented provides an appropriate level of analysis for most design problems that will be encountered for engineered channels. The design procedure assumes a rigid channel and is valid for both subcritical and supercritical flows. Applications involving rivers and large washes which do not function as a rigid, fixed-bed conveyance, require special design skills, and the design of these channels should not be attempted with the design techniques contained in this chapter. Channel linings of concrete, soil cement, riprap, graded earth, and grass are discussed in the manual. The analysis of natural channels is discussed in broader terms than is the treatment of engineered channels. Although the basic theory is the same for both channel types, more complex flow conditions (nonuniform and unsteady flow) and concepts of sediment transport often need to be incorporated in the analysis of natural channels.

1.2.9 Hydraulic Structures

The hydraulic structures that are described in Chapter 9 are used to control or alter flow characteristics, such as velocity, depth, energy, and to affect a change in the configuration of an open channel, such as channel slope. The purpose of such structures is to achieve safer and more stable conveyance systems with improved maintainability. Channel drop structures are a major topic of this chapter and guidance is provided for the design of various grade control structures. Information is provided for the dissipation of energy at conduit outlet structures with emphasis on riprap protection for outlets with moderate flow conditions and concrete structures for more severe conditions. Guidance is provided for the design or evaluation of channel transitions, bifurcation structures, spillways, trash racks, access ramps. A brief discussion is provided on groins and guide dikes. The design of various energy dissipators is included.

1.2.10 Detention Basins

Chapter 10 presents the engineering methodologies and details associated with the planning, analysis, and design of stormwater detention basins. Detention basins are man-made storage facilities that reduce peak discharges, thereby allowing downstream conveyance facilities to be reduced in size. Under certain conditions, they can also reduce the volume of stormwater runoff and mitigate the effects of urbanization. The theory and procedure for performing routing of an inflow flood through such facilities is provided.

1.2.11 Debris Barriers and Basins

Chapter 11 provides design guidelines for debris basins and debris racks/posts. The purpose of debris basins and barriers is to reduce the potential for debris clogging downstream channels, pipes, and culverts.

1.3 DISTRICT POLICIES

1.3.1 Standard of Care

For all design, engineers shall exercise "Standard of Ordinary Care" as would be expected of members of the civil engineering 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 District's policy that design may be considered to have a "Standard of Ordinary Care" if the design follows the Standard Drawings, Hydraulic Design Manual and/or other approved references, or has approved deviations from these documents, which are supported by sound engineering judgement or safety considerations.

In cases where strict adherence to the standards of design would be impractical or unreasonable, deviations may be approved providing that they comport with good engineering practice, ensure public health and safety, and clearly represent a more practical and reasonable solution. Any deviations from the standards of design shall be clearly identified by a note on the plans and approval of these plans shall constitute approval of such deviations. All deviations shall include supporting documentation and justification. Special circumstances, which may be cited to justify deviation from the standards, include, but are not limited to, alternative means of storm flow protection, environmental considerations, physical constraints, existing nearby uses, and economic considerations.

Engineers and planners should also consider that certain facilities might require a higher level of protection than the minimum protection levels set forth in this manual. Occasionally, agencies or organizations may require that critical infrastructure or emergency facilities have additional protection so their functioning will remain intact during large flood events. Such extraordinary protection levels will be considered on a case-by-case basis at the discretion of Riverside County or governing agency.

The respective roles and responsibilities pertaining to the operation and maintenance of stormwater management facilities within unincorporated areas within the District's service area is based on the current Memorandum of Understanding with Riverside County Transportation Department (March 2020, et seq.). This document also serves as the model for establishing the respective roles and responsibilities for the operation and maintenance of stormwater drainage facilities located within the incorporated Cities.

1.3.2 List of District Manuals

Although this manual is meant to be comprehensive within its' scope, the following Riverside County Flood Control and Water Conservation District (District) resources may be of use for the planning and design of stormwater management facilities. Available at https://rcflood.org/engineering-tools.

- District Hydrology Manual
- District Standard Plans
- District Drafting Manual

The Low Impact Development (LID) Best Management Practices (BMP) Handbooks for Santa Ana, Whitewater and Santa Margarita watersheds are available at https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/.

Please keep in mind that these resources are periodically updated. All references to District resources within this document refer to the most recent edition. Other resources referenced throughout the document may be consulted to obtain more detailed information on specific topics.

1.3.3 Use of Standard Drawings

This manual incorporates by reference the Riverside County Flood Control and Water Conservation District Standard Drawings, and designers may assume that these standard drawings are compatible with the guidelines presented in this manual. When there is no District Standard Drawing available for the design situation, possible additional resources for standard drawings include the American Public Works Association (APWA) for Southern California, the California Department of Transportation (Caltrans), Los Angeles County Department of Public Works, San Diego Regional Standards Committee, and Orange County Public Works. Other resources may also be used for the design of stormwater drainage and flood management facilities, subject to review and approval by the District or local governing agency.

1.3.4 District Accepted Computer Applications

Currently accepted computer applications are provided rcflood.org>Business>Engineering Tools>District Accepted Software. Computer applications other than those listed on the District website require prior approval of the District. In these cases, supporting documentation may be required at the discretion of the plan checker. Additional review fees may also be required for the District to hire personnel with specialized expertise to perform a thorough evaluation.

1.3.5 Safety

During storm events, people have been known to intentionally or inadvertently enter areas that become extremely dangerous during flood conditions. Or, worse, they may purposely try to boat or float in stormwater management facilities during high runoff levels. It is not possible to construct stormwater management facilities that are completely without hazard, that will completely preclude people from engaging in high-risk behaviors, and that will also be hydraulically efficient. However, reasonable levels of protection can be provided so that people exercising reasonable judgment are protected, even when the structure is performing its primary function, i.e., efficiently storing or transporting stormwater runoff.

An overriding goal of any District maintained stormwater management facility is to protect, maintain, and enhance the public health, safety, and general welfare by controlling the adverse effects of stormwater runoff.

The issue of safety includes the following principles:

- Stormwater naturally accumulates in and flows through both developed and undeveloped areas, frequently in amounts that present hazards to property, traffic, life, and health. Protection of public safety (life and health) is the highest priority.
- Because of the accumulation and/or movement of stormwater, certain safety hazards are unavoidable.

Public access and safety concerns are inherent elements in the design of all stormwater management facilities. These elements are of primary importance, particularly in the case of multiple-use facilities where public use is allowed in areas that are periodically subject to flooding, e.g., large detention basins and "green-belt" channels. The safety considerations associated with stormwater storage or conveyance facilities are user education, advance warning, potential water depth/velocity, facility slopes where ingress and egress is anticipated, escape routes from flooded areas, and draw-down time.

Appropriate safety mitigation measures can be addressed in two ways. The first relates to the need to identify and communicate potential hazards to the public. For example, with proper signage, users can be made aware of the existence of potential hazards, such as flooding, high velocity flows, etc. User education is a fundamental element in safety design for a stormwater facility. Clear, concise signage with illustrative graphics can inform the public of the primary flood control purpose of the facility and describe the various features and their potential dangers when floodwaters are present.

The second relates to the design of the facility to include safety devices that can be readily maintained. Appropriate steps should be taken to mitigate potentially dangerous conditions. Where the dangerous condition cannot be mitigated, appropriate measures should be implemented to keep users away from hazardous locations.

Safety devices can be divided into two types, as follows:

- ➢ Devices that Limit or Deter Access:

- Fencing
- Guard rails
- Warning signs
- Safety barriers

➢ Devices that Permit Escape:

- Safety racks (to prevent persons already in a flood hazard from passing to an area of more severe hazard)
- Egress facilities (mild slopes, stepped walls, ladders, etc.)
- Safety nets and cables

An important distinction between these two categories is that devices that permit escape may also impede the flow of stormwater into or through stormwater management facilities.

Safety devices for stormwater management facilities should be considered for both dry weather and runoff conditions. Dry weather hazards include traffic and personal safety. Examples of traffic hazards include improper placement of guardrails on structures, unprotected drops at structures located near roadways, and grading, all of which may contribute to vehicle accidents. Dry weather hazards include vertical drops or walls that may present hazards to the public and which may serve as an attractive nuisance.

The basic concept of this approach to safety is to apply more restrictive measures as hazard levels increase. The primary purpose for constructing stormwater management facilities is to provide for the efficient collection and conveyance of stormwater to minimize property damage and to permit traffic flow across and parallel to drainageways; therefore, safety in this context refers to protection from life and health hazards.

1.4 RECREATIONAL USES

The mission of the District is to responsibly manage stormwater in service of safe, sustainable, and livable communities. To accomplish part of the mission, the District operates and maintains a network of flood control facilities that are essential for the protection of the public health and safety in western Riverside County.

Because the flood control hazard associated with these facilities is periodic in nature, it is the policy of the District to allow other entities, such as Cities and/or Parks and Recreation Districts, opportunities to develop compatible non-motorized recreational uses within District properties, such as equestrian, walking, jogging, or bicycling, as part of a publicly maintained park and/or trail system. **Coordination with the District's Operations, Maintenance, and Regulatory Divisions early in the planning stage is critical** to ensure that the proposed project features are designed in a manner to be compatible with the District's mission and can be feasibly implemented and maintained.

These potential recreational uses principally apply to open channels and basins.

1.4.1 Trails

The District may permit the inclusion of public-use trails along District-maintained facilities, where such trails are maintained through a license agreement by other public entities, such as Cities and/or Parks and Recreation Districts. Acceptable trail uses include compatible non-motorized recreational uses, such as equestrian, walking, jogging, and bicycling (class 1 and 2 electric bikes per State Assembly Bill 1096 are permitted), on/within District rights of way as part of a regional trail system. Coordination with the District's Planning, Operations and Maintenance, and Regulatory Divisions early in the planning stage of a proposed project is critical to ensure it is compatible with the District's mission and can be feasibly implemented. The following is a general outline of what is needed in order to establish a public trail within the District's rights of way:

1. The primary function and purpose of the District rights of way is intended for flood control operations. All proposed trail improvements come second to this primary function.

2. Trails can only be established where the District has ownership of the property. In some cases, the District may only hold easement rights, therefore, the Applicant (City, Developer, Parks and Rec District, etc.) proposing the trail must also secure the consent of the underlying fee owner. Trails must be operated and maintained by a public entity who shall be responsible for maintaining all aspects of the installed trail improvements, including, but not limited to, the removal of weeds, trash, and graffiti. Private entity ownership and/or maintenance of a trail system is not acceptable on District maintained facilities.

3. The Applicant proposing a trail within District right of way must develop a Trail Plan that clearly identifies the location of the trail, public access points, and what District facilities will be impacted. The Trail Plan should clearly address and show any proposed improvements within the District's right of way, such as fencing, paving, shade structures, benches, access paths, signage, vegetation, irrigation, utilities, and planting details. A list of accepted and prohibited plants is also provided here. Improvements within District rights of way cannot obstruct or interfere with the District’s ability to perform its primary function and all improvements shall be constructed to ensure sufficient structural integrity for the District’s heavy equipment, as approved by the District. See Attachment A – Trail Plan Design Considerations for more information at https://rcflood.org/obtain-encroachment-or-access-permit.

4. As part of the Trail Plan, the Applicant shall prepare, circulate, and adopt/certify the appropriate CEQA document (Notice of Exemption, Negative Declaration or Environmental Impact Report). This document should address all direct and indirect impacts, including any cumulative and/or reasonably foreseeable impacts associated with establishing the proposed trail on or within District facilities or rights of way in accordance with current CEQA guidelines. The District will be a Responsible Agency on any trail that is proposed on or within District facilities or rights of way and should be listed as such in the project’s CEQA document.

5. Once the Trail Plan and associated CEQA document has been adopted by the Lead Agency, the Applicant can apply for an Encroachment Permit (EP) from the District. The EP application can be found here. Any improvements to be constructed for the establishment of the trail,

such as those listed in #3 above, should clearly be identified in order to be approved under the issuance of the EP.

6. Additionally, the Applicant proposing to establish a trail within District Right of way will need to submit an application for an agreement preparation (Attachment B). The application can be found at Attachment B.

   The Agreement establishes responsibility for the different components and authorizes the use of the District's right of way for public trail purposes. The public entity responsible for maintaining trail will need to indemnify the District from any liability regarding the public's use of the trail. An initial deposit of $10,000 will be required in addition to the EP deposit and shall be included with the EP and agreement application. This is a deposit-based fee that will need to cover the cost of reviewing the EP application, associated CEQA and MSHCP compliance documentation, improvement plans, regulatory permits (if needed), preparation of the EP and agreement, as well as inspections during construction. See Attachment B.1 for a sample license agreement for a specific facility and Attachment B.2 if the District has a master license agreement with the City. Attachments are available on the District's EP website: Obtain Encroachment or Access Permit | Riverside County Flood Control and Water Conservation District (rcflood.org)

1.4.2 Parks

Recreational uses, such as a park, may be proposed within District-maintained basins. The primary function of a basin is for flood control use, therefore, there is a need for storage and expectation of flooding. The Applicant will generally be allowed to install non-structural items such as landscape and hardscape (no buildings, no restrooms,) as long as there is a public entity (Licensee) willing to enter into a license agreement to take on maintenance responsibilities of such items with the understanding that the basin may at any time be affected by storms that could impact the recreational features. The Licensee shall repair/rebuild any park features as agreed upon in the license agreement. Parks proposed within a District-maintained basin will have similar considerations as listed in Attachment A for trails, but will need to be considered on a case-by-case basis based on the specific scope and nature of uses within the park. Close and early coordination with the District and the entity proposed for maintenance is required before proposing a park within a District-maintained basin.

1.5 DRAINAGE LAW

Case law concerning drainage is too extensive for a comprehensive discussion in this manual. However, there are many legal principles that might affect landowners and engineers involved in the design, construction, and maintenance of stormwater management facilities. California drainage law is based on the following "good neighbor" policies of what constitutes reasonable action (for example, Keys v. Romley, 1966 et al.).

- Landowners have the right to discharge storm water in a reasonable manner.
- Downgrade landowners have an obligation to accept landowners' naturally flowing surface waters.

- Landowners must take reasonable care to avoid damage to adjacent properties due to runoff.

Many court decisions regarding drainage law have been based upon these principles. For instance, it is generally accepted that upgrade landowners may not increase the volume or velocity of surface flows to the detriment of downstream landowners. Readers of this manual are encouraged to develop a familiarity with drainage law principles and incorporate "good neighbor" policies into their stormwater management facility design.

1.6 AUTHORITY AND JURISDICTION OF RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT

The Riverside County Flood Control and Water Conservation District (District), established July 7, 1945 under authorization of Act 6642, Chapter 1122 of the State of California Statues of 1945, was created to protect watercourses, watersheds, public highways, life, and property within the unincorporated areas of the County from damage caused by storm and floodwaters. Further legislative authorizations for the District and its activities pertaining to stormwater drainage and flood management can be found in the following sources:

State of California Subdivision Map Act

- Division 2 Chapter 4 Article 1 §66474.7
- Division 2 Chapter 4 Article 5 §66483
- Division 2 Chapter 4 Article 6 §66488

Riverside County Code of Regulatory Ordinances

- Ordinance No. 458 Regulating Special Flood Hazard Areas
- Ordinance No. 671 Fees for Land Use and Related Functions
- County of Riverside Ordinance No. 348 Land Use
- County of Riverside Ordinance No. 460 Regulating the Division of Land

1.7 SUPPLEMENTAL DISCIPLINES

1.7.1 Geotechnical Engineering

Geotechnical investigations may be required for designs of various stormwater management facilities. Determination of foundation characteristics and evaluation of soil materials proposed for construction is routinely required for many drainage projects. Samples obtained from borings and exploratory pits should be tested under laboratory conditions to evaluate more precisely the soil and rock classification properties, strength, permeability, compatibility, and other specialized parameters pertinent to the specific project conditions. The results of these laboratory tests and associated analyses are to be used to develop guidelines for safe and economic designs. This Hydraulic Design Manual does not go into the requirements and procedures for geotechnical studies. Nonetheless, the designer must always consider the importance of this information and secure this expertise as appropriate for the project at hand.

1.7.2 Structural Engineering

Structural engineering expertise is required in applications where standard details (e.g., District, Caltrans, etc.) do not meet the project's needs. Here, the structural engineer must assess the anticipated loads or forces that the drainage structure must withstand and specify the material, strength, and geometry for the structure. This Hydraulic Design Manual does not provide guidance for structural analysis. When the design engineer faces situations where the available standard details cannot be applied or there is reason to doubt the applicability of a standard, a structural and/or civil engineer must design the drainage improvement for structural integrity.

1.7.3 Environmental/Cultural Resources Expertise

Stormwater management facilities often co-exist, interact, or have the potential to affect cultural or natural resources. The designer of drainage improvements must consider these factors during the design process. Depending upon the project at hand, specialty studies related to cultural resources, waters of the U.S., historic properties, wildlife, hazardous waste, etc. may be required. With recognition of these various issues, the designer must realize the need for a specialist(s) to assist with the design of the stormwater drainage improvement/project. Often, alternative alignments or configurations may be required to avoid or mitigate impacts to these resources. This manual does not delve into these resource issues nor provide guidance as to their mitigation.

1.7.4 Water Quality Expertise

Detention facilities may also function as multi-use facilities to meet water quality performance standards. For water quality performance standards including pollutant control and hydromodification, the Design Engineer is directed to the Water Quality Management Plans (WQMPs) for projects located in the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable. For design guidance for best management practices (BMPs) to meet water quality performance standards, the Design Engineer is directed to the Design Handbook for Low Impact Development Best Management Practices (LID BMP) for the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable. Additionally, for transportation related projects that include the construction of new or the retrofit of existing transportation surfaces including sidewalks, bicycle lanes or trails located within the Santa Margarita and Santa Ana Regions of Riverside County, the Design Engineer is directed to Section 5.4. Where earthen channels may also function as stream restoration / rehabilitation channels, the Design Engineer is directed to Section 8.4.1.3.

For multi-use detention facilities to be maintained by the District, refer to guidelines in Chapter 10 . Otherwise, guidelines for access roads and ramps and sediment forebays as noted in the Design Handbook for LID BMP should be considered. Use of impervious features within the facility may trigger additional pollutant control and hydromodification requirements. Infiltration guidelines are provided in the Design Handbook for LID BMP. Interior basin cut/fill slopes shall be 3:1 or flatter to ensure public safety. Pollutant control and hydromodification basins shall drawdown within 72 hours. Combinations of outlets may be used to also mitigate increased runoff, pollutant control and hydromodification impacts, where feasible. Both increased runoff and hydromodification analyses shall be conducted, as applicable. Compliance with each performance criteria shall be determined and documented independently. Pollutant control and hydromodification volumes, flowrates and velocities shall be documented on multi-use detention facility plans. Maintenance of multi-use detention facilities shall also ensure continued functions of the multi-use detention facility. Any overlapping objectives shall be coordinated between the storm water management engineer and the flood control engineer.

1.7.5 Utilities

Since stormwater management facilities often co-exist, interact, or interfere with the built environment, permission, and/or coordination with other agencies may be necessary when building new facilities crossing existing utilities (e.g., large water lines, etc.). Avoiding conflicts with existing utilities is always preferable, where possible, however relocations of conflicting utilities may be required in some cases to accommodate the stormwater management facilities.

1.8 REFERENCES

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Riverside County Flood Control and Water Conservation District Design Handbook for Low Impact Development Best Management Practices, September 2011. https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/

Riverside County Flood Control and Water Conservation District Santa Margarita River Watershed Region Design Handbook for Low Impact Development Best Management Practices, June 2018. https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/

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Riverside County, March 2020, Memorandum of Understanding Riverside County on Behalf of its Transportation Department and Riverside County Flood Control and Water Conservation District for Design, Construction, Inspection and Maintenance of Flood Control Drainage Facilities. MOU Transportation RCFC

U.S. Department of Transportation (USDOT), Federal Highway Administration (FHWA),1984, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12. Publication No. FHWA-TS-84-202

——, 2012, Hydraulic Design of Highway Culverts, Hydraulic Design Series No.5. Publication No. FHWA-HIF-12-026

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2 SUBMITTAL REQUIREMENTS

2.1 GENERAL

This chapter addresses the District's requirements for entitlement and plan check submittals. When District acceptance for operation and maintenance of stormwater management facilities is intended, sufficient information in the form of maps, calculations, drawings, and construction specifications for the facility is required. All construction and materials shown on plans shall conform to all applicable specifications provided in the District/Transportation Department MOU specification at MOU.

Engineering calculations shall be prepared in accordance with the resources and references presented in this manual, the Riverside County Hydrology Manual, and any addendums or revisions thereto. The design engineer's signature with license stamp shall be included on final plans, reports, and calculations. The plan check application is available from the District's website at rcflood.org>Services>Submit for Plan Check. The review fee is deposit based on time charged for the review. The plan check review cost will be dependent upon the number of reviews required to address correction comments.

Prior to the formal submission of an application and prior to the applicant spending time and resources preparing detailed plans and technical studies, the applicant can use the Development Pre-Application Review (PAR) process. During the PAR process, the project exhibit is distributed to various County departments for review. Each department will provide general input, observations regarding the project and items to address prior to submitting the application formally. The PAR helps shorten the formal review period, and in turn allows the project to go to a public hearing quicker. Application (including forms and filing instructions) are available on the Planning website https://planning.rctlma.org/Development-Process. District staff are available for cursory pre-design and pre-submittal consultation for the applicant to present a conceptual plan, troubleshoot project issues for potentially complex development proposals, and confirm the project meets the District's design and maintenance standards outlined in this manual.

2.2 ENGINEERED PLANS

All plan sheets and drafting shall conform to this manual and the latest edition of the District Drafting Manual. For facilities to be maintained by the District, early consultation with District staff is recommended before setting the final alignment. Construction drawings of District facilities shall be prepared on plan sheets (typically 22"x34") to allow a 50% reduction to 11"x17" for reproduction, microfiche, and filing. Sizes other than these require governing Agency approval.

2.3 HYDROLOGY AND HYDRAULICS (DRAINAGE) REPORT

A Hydrology and Hydraulics (H&H), aka Drainage, report shall include a complete project description, project setting including discussion of pre and post project, interim condition, existing and ultimate land use, any drainage issues related to the site, a summary of the findings or conclusion, offsite hydrology, onsite hydrology, hydraulic calculations, and a hydrology map. Complete supporting information and materials must be included in the report, including all charts, tables, graphs, land use maps, and soil maps used to determine initial time of concentration, hydrologic values, and other modeling parameters.

The following are requirements for drainage reports which are to be reviewed and approved by the District.

2.3.1 General

1. The final report must be signed and stamped by the registered civil engineer who is in responsible charge of preparing the drainage study. The report should be labeled "draft" for initial submittals.
2. The report must be organized in a logical manner, and a summary of the results and associated impacts resulting from the project must be given in the narrative portion of the report. The preferred report format is shown on Figure 2.1.
3. The report shall include information necessary (i.e., Tract No., street name, facility name) for convenient filing so that can it be easily recalled in the future.
4. If electronically submitted, a document with active bookmarks is suggested.

2.3.2 Hydrology Calculations

The hydrology section of the drainage report shall include hydrology calculations based on the District's Hydrology Manual and its addenda. All requirements in the project's Conditions of Approval shall be met. The section shall include computer modeling input and output and/or hand calculations, and a hydrology map and any other supporting exhibits to facilitate review by the plan checker. The onsite and offsite flowrate results shall be summarized in the report. Additionally, mainline hydrology and catch basin hydrology should be separated with drainage maps for each portion and delineation for each inlet. Hydrology studies for land development projects shall analyze both existing condition (i.e., pre-project, current land use), interim condition (i.e., existing, ultimate land use, as applies), and ultimate condition (i.e., post-project, ultimate land use, buildout). In addition, fully discuss, document, and analyze any interim drainage conditions, when applicable.

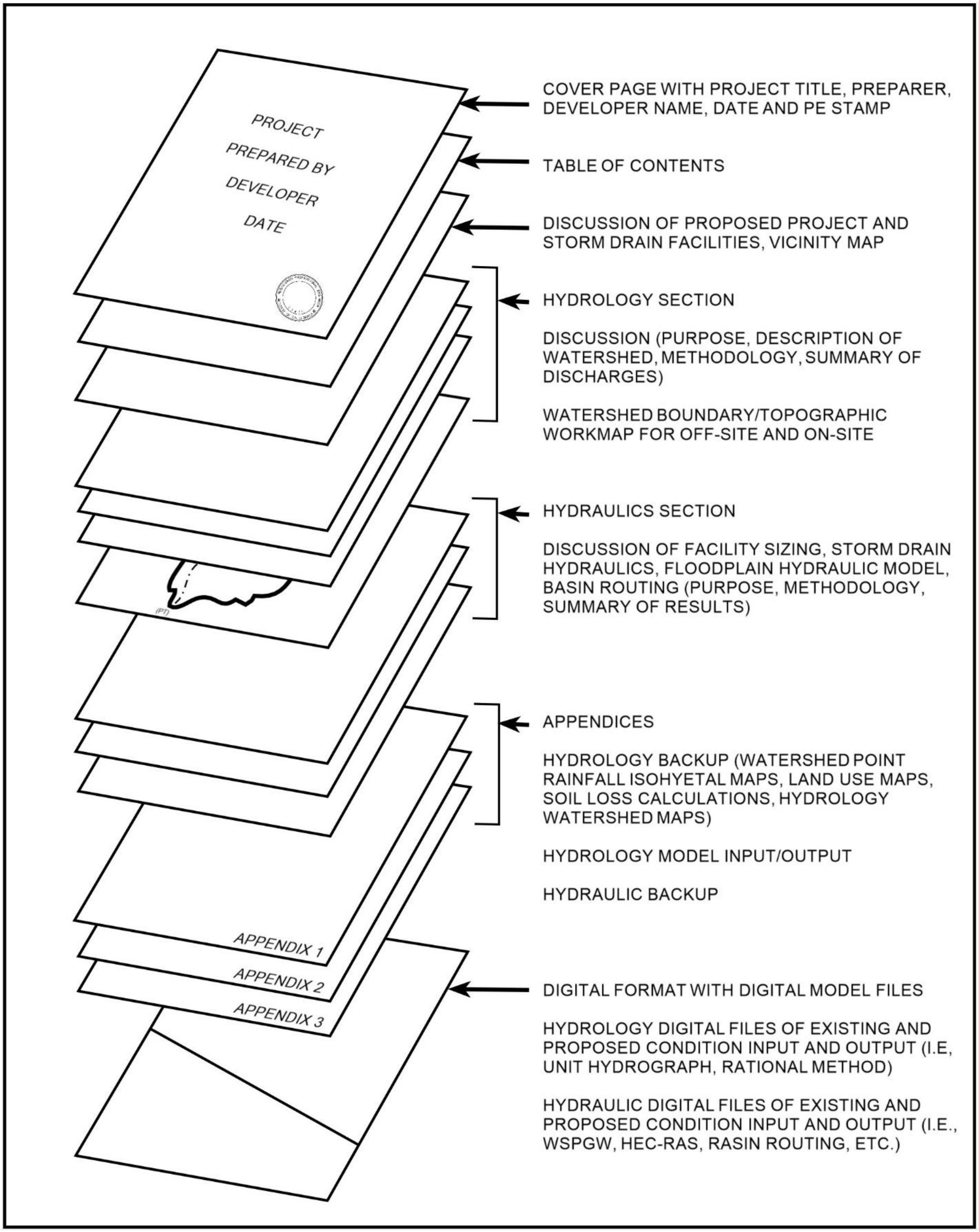
2.3.2.1 Hydrology Chapter Outline

A suggested table of contents for the hydrology chapter of the drainage report is provided below:

1. Overview
2. Source Data
   a. Topography
   b. Rainfall
   c. Land Use
   d. Soils
3. Mainline Hydrology
   a. Method
   b. Drainage Areas (offsite and onsite)
   c. Results
4. Catch Basin / Inlet Hydrology
   a. Method
   b. Drainage Areas to each inlet (or group of inlets)

c. Results

Figure 2.1: Typical Report Format for Drainage Report



2.3.3 Hydraulic Calculations

The ultimate hydraulic grade line (HGL)/water surface profile of the drainage facility shall be calculated and plotted on the plans. In those instances where an interim condition HGL exceeds that of the ultimate HGL, both HGLs shall be calculated and plotted on the plans. The ultimate and interim HGL plotted on the plans should be based on the 'capacity' criteria described in Section 8.3.4. Drainage report submittals shall include narrative, boundary conditions and other assumptions, digital versions of all computer modeling input and output and/or hand calculations, mainline, lateral, and catch basin hydraulics, reference drawings (as-builts), mapping and/or other supporting exhibits necessary to facilitate review by the plan checker.

2.3.3.1 Hydraulic Chapter Outline

A suggested table of contents for the hydraulic chapter of the drainage report is provided below:

1. Mainlines
    a. Mainline #1
        i. Design flow rate
        ii. Description of system
            1. Conduit types and materials
            2. Sizes
            3. Upstream and downstream facilities or watercourses
            4. Boundary conditions
        iii. Method of Analysis
            1. Software, if used
            2. Confirmation of channel lining suitability, rock sizing, etc.
        iv. Results
    b. Mainline #2
2. Inlets and Connector Pipes
    a. Mainline #1
        i. Method of analysis
            1. For the interception capacity of the various inlet types
            2. For the connector pipes
        ii. Table summary for type, size, interception, and bypass for each inlet draining to Mainline #1. ID numbers should be used to correlate catch basins, connector pipes, and the plans.
        iii. Table summary of hydraulic calculations for the connector pipes entering Mainline #1. ID numbers should be used to correlate catch basin, connector pipes, and the plans.
    b. Mainline #2
3. Other Hydraulic Structures
    a. Hydraulic jump
    b. Debris rack
    c. Energy dissipator
    d. Etc.

2.3.3.1.1 HEC-RAS Content – Work Maps

Content – 1D Hydraulic Model Work Maps

Complete and separate existing and proposed condition work maps must be provided. The work maps must include the following:

1. Cross sections must be drawn at the proper length and include a layout line coinciding with the channel or floodplain centerline. The layout line shall be labeled with stationing on the work maps.
2. Each cross section must be labeled with the cross-section number. The label must be located at the left end of the cross section looking downstream. The right and left "overbank" locations on each cross section shall be marked on the work map in some consistent fashion (e.g., a circle, square, or triangle must be placed on the cross section at each overbank location).
3. The pre-project and post-project floodplain limits/extents must be accurately plotted on each work map. In situations where the project results in proposed water surface elevation increases, a single composite floodplain map showing both the existing and proposed floodplain extents must be provided.
4. Areas modeled as ineffective flow must be clearly shown.
5. The underlying topography and contour labels on the work map must be legible. Work map must include topography date, horizontal and vertical datum used, and benchmark.
6. All proposed grading, culverts, bridges, drop structures, access ramps, etc. that impact the floodplain must be shown.
7. Proposed access roads and turnarounds must be shown.
8. Proposed finished grade elevations adjacent to the floodplain must be shown. When grading for proposed conditions is shown on/over existing topography, sufficient labeling of grades at top and toe of slopes and flowlines shall be provided to allow the reviewer to reconcile the model cross sections to the work maps.
9. Legend, scale, and north arrow shall be shown on all maps. Ensure that the return period and duration are stated on each work map where the floodplain is plotted.
10. Work map must be signed and stamped by a registered civil engineer.

Content – 2D Hydraulic Model Work Maps

Complete and separate existing and proposed condition work maps must be provided. The work maps must include the following:

1. Topographic contours, proposed grading, and planimetric features that impact the hydraulic model (e.g., roads, bridges, levees, buildings, etc.) must be plotted. Index contours for both existing topography and proposed grading are to be labeled legibly for the anticipated map plot scale. Contours must be generated from the submitted terrain files.
2. Hydrologic boundary condition (inflow and outflow) locations and tie-in locations to adjacent hydraulic models (if any) must be identified either on the floodplain maps or on a standalone work map. At each boundary location, indicate the peak flow rate of the hydrograph and whether it is flowing into or out of the model (e.g., Q_{in} = 3,000 cfs).

3. Maximum floodplain extents must be plotted along with water surface elevation contours not to exceed 4-foot intervals. The index water surface elevation contours are to be labeled legibly for the anticipated map plot scale.
4. For models supporting a development project or where the impacts of proposed changes are being analyzed, a depth-difference grid shall be developed by subtracting the pre-project depth from the post-project depth and from this result, a color-coded floodplain map shall be developed indicating increases versus decreases in water surface elevation due to development (or other proposed changes). Note that for this to work as intended the pre-project and post-project mesh outside of the project extents must be identical.
5. A representative channel/floodplain centerline is to be included on the work map.
6. A color-coded Manning's n value work map of the 2D model mesh is to be provided.
7. Legend, scale, and north arrow shall be shown on all maps. Ensure that the return period and duration is stated on each work map where the floodplain is plotted.

2.3.3.1.2 HEC-RAS Content – Narrative and Supporting Information

A complete hydraulic study must be provided including:

1. A printout of HEC-RAS input and output files for both pre-project and post-project conditions must be provided. For 1D models, the output files shall also include at least one summary table. For unsteady models (1D and 2D), the output files shall also include the computational log file demonstrating that the volume gained or lost during the analysis is less than 1% of the total model volume.
2. The report must include a narrative and, if multiple scenarios are modeled, a table explaining the modeling runs. There are often multiple "Projects" and "Plans" employed. The names and interrelationship of all component input files and output files shall be explained.
3. All proposed grading, culverts, bridges, drop structures, access ramps, etc. that impact the floodplain must be included in the modeling and discussed in the narrative.
4. A flash drive, CD or other digital format with the input and output files shall be included in the report package. HEC-RAS provides an option to select which files are zipped together. The District requests that only applicable plans be selected under "Base RAS Input Files", all "GIS Input Files", and all "Computed Files".
5. For 1D models, a summary table including cross section station, 100-year flowrate, and water surface elevation shall be provided. Where a 1D model is supporting a development project, the pre-project and post-project water surface elevations and velocities, and difference in water surface elevation and velocities shall be provided at each cross section. The table may be included in the report narrative or on the work map. In either case, the report narrative shall discuss the results presented on the summary table.

6. For 1D models, a channel centerline profile plot showing thalweg elevation, water surface elevation and critical depth must be provided. The thalweg need not necessarily coincide with the layout line.

7. For 1D models, cross section plots for all cross sections plotted at no more than four per each 8½ x 11 sheet must be provided. Each cross section must be labeled with the cross-section number and must show the entire cross section, water surface elevation and distribution of Manning's " n" values over the cross section. If feasible, it is greatly preferred that the scale of the cross-section plots be consistent.

8. A description of the Manning's n values used and an explanation for why they were chosen must be provided. Where n values are based on a database, the source and date of this information shall be provided.

9. A description of and a rational explanation for how the upstream and downstream boundary conditions were established must be provided in the report narrative.

10. Photographs of the existing study reach should be included if it is felt they will assist in conveying information to the reviewer.

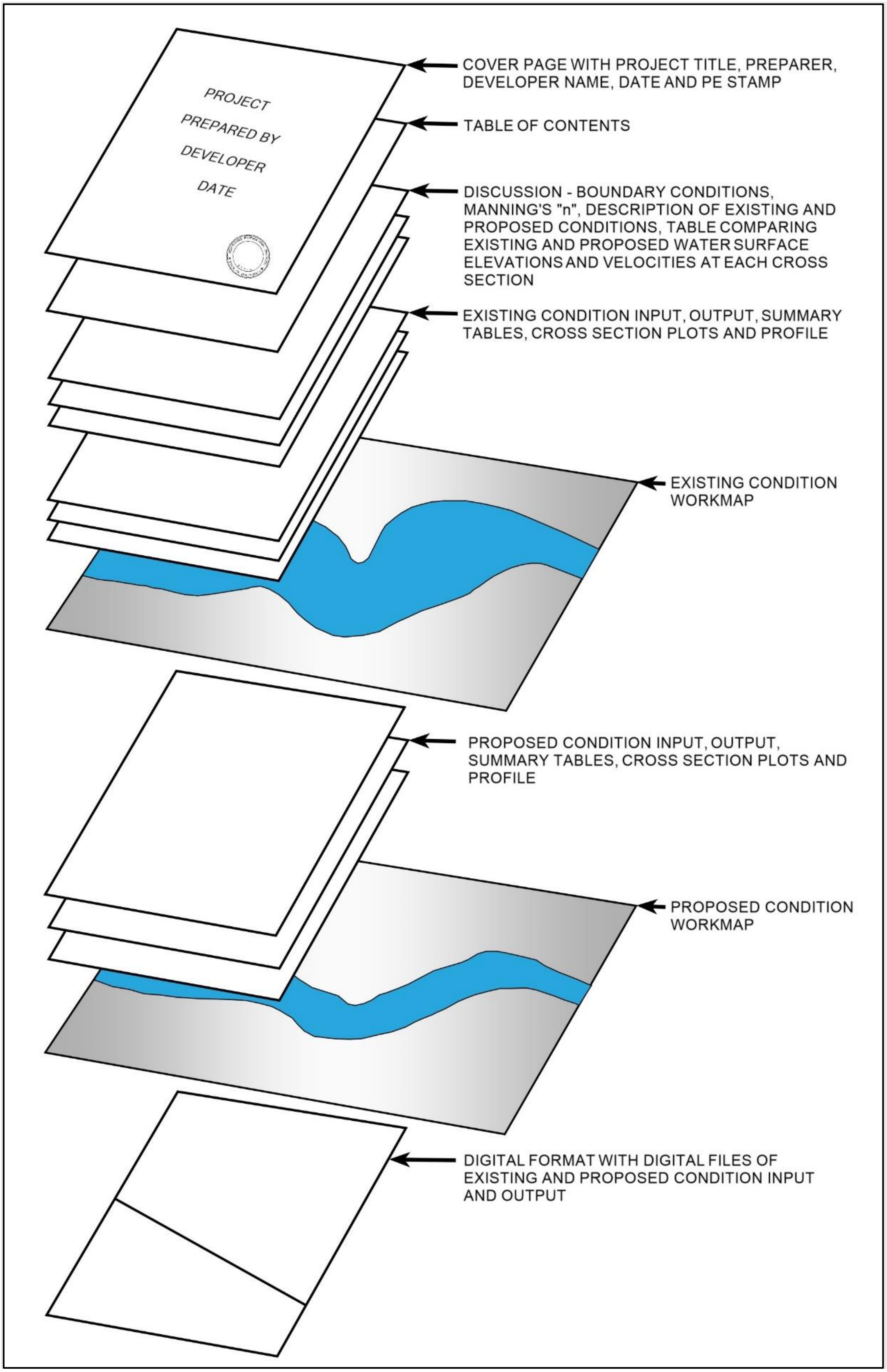
11. Where rating curves are utilized to describe a boundary condition or hydraulic structure, the rating curve backup calculations and a narrative describing the rating curve development shall be provided.

12. For 2D models each terrain model shall be identified and described along with the constituent parts of that terrain model (e.g., individual TIF files that comprise the terrain model). Each of the constituent parts are to be listed in order of prioritization, along with a description of that constituent part and the source (e.g., topographic field survey by Sam's Surveying dated June 23, 2021). Where topographic field survey or LIDAR data is utilized, a statement regarding the accuracy shall be provided. Changes between the existing and proposed condition terrain models are to be described in the narrative.

13. For 2D models, where the impacts of proposed project are being analyzed, the increase in water surface elevation and velocity shall be identified on the 2D Hydraulic Model Work Map. The report shall discuss the input and output results.

14. For 2D models, a description of the sizing of the mesh cells, break lines, computational timestep and relation to Courant Number, and computational method (i.e., Diffusion Wave or Shallow Water equations) chosen for the modeling and an explanation for why they were chosen shall be provided.

Figure 2.2: Typical Report Format for HEC-RAS Study



2.3.3.2 WSPGW Hydraulic Calculations General Requirements

WSPGW: Water Surface Pressure Gradient for Windows software may be utilized for hydraulic analysis of closed conduits and prismatic channels. The program computes and plots uniform and non-uniform steady flow water surface profiles and pressure gradients with irregular or regular sections.

2.3.3.2.1 WSPGW Computational Procedure

The channel or conduit system is divided into the following elements: system outlet, reach, transition, junction, bridge exit, bridge entrance, wall entrance (sudden contraction), wall exit (sudden expansion), and system headworks. Each element is internally assigned a number and the user provides the station and invert elevation for each element. The program determines the distance between elements by the difference between stations. Additional intermediate "points" between elements are internally set by the program whenever the velocity head between two points exceeds 10 percent. Up to 200 additional points may be set between elements. The input data must consist of at least three elements (system outlet, any other element with distance, and the system headworks). A maximum of 200 elements is allowed. A greater number of elements requires the user to make separate data files for the analysis.

Additional description of the basic theory, input data elements, sequence of input and program operation, computational procedures and equations, plan and profile drawing, or program limitations can be found in the User's Manual of the WSPGW Program.

2.3.3.2.2 Water Surface Controls

Water surface elevations at the system outlet and system headworks (starting or initial water surface elevations) are typical input values. If not specified, the program uses critical depth controls to begin computations. If a water surface is given at the system outlet, the program will use it only if the energy grade line of the computed channel is less than the water surface elevation given. The design engineer shall confirm appropriate assumptions to set appropriate water surface elevations at the system outlet and headworks.

2.3.3.2.3 WSPGW Content - Hydraulic Calculations

After successfully running the WSPG program, a printout and digital version of all input data files (.WSW), edit output files (EDT), and final output files (OUT) shall be submitted for review. The Hydraulic Grade Line shall be plotted on the construction drawings profiles, and hydraulic data (flow rate and maximum velocity for each reach) shall be added to the profiles.

Construction Drawings plans and profiles shall have sufficient stationing, elevation, and element data to support the submitted WSPGW calculations. The recommendations of the District's Drafting Manual shall be followed in the preparation of the construction drawings.

2.4 STRUCTURAL CALCULATIONS

The structural design report must 1) for any standard drawings used (no custom structural calculations) confirm that the site conditions and design match any structural assumptions or limitations on each of the standard drawings they are using, and 2) for any custom-designed hydraulic structures, detail the codes used, load cases, etc. Structural calculations shall be presented within a signed and sealed report that includes: cover sheet, detailed design criteria, soil data, geotechnical report recommendations, and structural details or sketches as needed to clarify the intent of the calculations. Submittals shall include computer model input and output and/or hand calculations that can be easily followed by the plan checker. The titles of structural calculation reports shall provide for convenient filing (e.g., Tract No., street name, facility name) that can be easily recalled for future reference.

2.5 GEOTECHNICAL REPORT

A geotechnical (soils) report specifically addressing requirements of facility design and construction shall be submitted with the calculations and plans. Submittals shall include computer model input and output and/or hand calculations, exhibits, and mapping that can be easily followed by the plan checker. Studies and reports shall provide for convenient filing (e.g., Tract No., street name, facility name) that can be easily recalled for future reference.

The Geotechnical Report should address at minimum, but is not limited to the following:

1. Overall feasibility of the project as proposed
2. Soil types/soil logs
3. Geologic setting/seismicity condition of the area
4. Excavation characteristics
5. Suitability of onsite materials for use as backfill, and any limitations
6. Description of groundwater, site, and subsurface conditions
7. Recommendations for unusual soil conditions or groundwater conditions during construction, if encountered
8. Soil compressibility, preliminary soil strength
9. Soluble surface analysis and corrosion protection requirements
10. Trench and shoring design criteria

Specific design parameters shall be provided for the pipe installation, including, but not limited to:

1. Allowable bearing pressure
2. Design lateral earth pressures
3. Shoring/trench safety
4. Coefficient of friction
5. Sand equivalent values and soil density
6. Site preparation including compaction requirements and compaction characteristic of native soil
7. Shrinkage and subsidence
8. Corrosion protection recommendations
9. Design parameters for specific structures

For drainage facilities using cast-in-place concrete pipe, see the current Memorandum of Understanding with Riverside County Transportation (March 2020) Section 5.3.2 for additional requirements.

2.6 CEQA AND REGULATORY PERMIT COMPLIANCE

All CEQA mitigation measures must not inhibit or prohibit maintenance of the system and regulatory permits must be obtained to fully enable such maintenance. Submittals of draft and final CEQA documents, as well as draft and final regulatory permits must be provided to the District for review.

2.7 ENTITLEMENT SUBMITTAL

Development projects being submitted to County Planning for discretionary entitlement review are reviewed by multiple departments within the County, including the District. The purpose of entitlement drainage review is to ensure drainage is addressed at a planning level to 1) protect the site from offsite flows in the existing, interim, and ultimate conditions, 2) accommodate right of way for proposed drainage facilities, 3) provide an adequate outlet, 4) not adversely affect adjacent properties, 5) properly floodproof structures, and 6) potentially mitigate for increases in runoff.

Development projects should consider the following items related to drainage:

- Preliminary drainage report or site exhibits should explain how the site is being protected from 100-year offsite surface drainage and how onsite drainage is being conveyed. Demonstrate adequate outlet for site discharge without adversely impacting adjacent and downstream properties.
- Increased runoff mitigation may be required, see Section 10.4 for additional information. Mitigation of the increase in peak 100-year flowrate may also be required if the proposed land use would produce more runoff than the downstream storm drain facilities were designed to convey. See Chapter 10 for 100-year detention basin guidelines.
- When possible, mapped flood hazard areas identified in Ordinance No. 458 should be avoided. If the proposed project is located within a mapped flood hazard area, identify impacts to flood hazard and prove proposed structures are adequately protected and provide adequate flow through area without impacting neighboring properties.
- Identify natural watercourses impacting the project site, as well as existing and proposed drainage facilities.
- Exhibits shall depict all proposed grading, including, but not limited to, all cut/fill slopes with slope ratios, pad sites, pad elevations, and finished floor elevations.
- Perpetuate existing natural drainage patterns with respect to tributary drainage areas, outlet points, and outlet conditions to avoid flow diversions.
- Identify maintenance responsibility of proposed storm drain facilities (e.g., culverts, basins, channels, etc.). Facilities to be maintained by the District must consider District design standards.
- Additional analysis and design details may be required prior to conditioning and approval to determine whether the proposed drainage solutions can be feasibly implemented without significant changes to the project.

2.8 PLAN CHECK SUBMITTAL REQUIREMENTS

2.8.1 Preliminary Design

The District understands that there may be some projects that will require a pre-design meeting to ensure the assumptions made by the engineer are acceptable and congruent with District standards and specifications. A pre-submittal meeting can be scheduled by contacting the Plan Check supervisor whose contact information can be found at https://rcflood.org/submit-plan-check.

For complex projects, the District recommends a preliminary design (30%) level submittal of the improvement plans and drainage study followed by a meeting to discuss the project issues. Complex projects may include, but are not limited to, any of the following features:

Drop structures
Lined side slope soft-bottom channels
Launch rock
Debris basins
Regional detention basins or dams
Levees
Modification of a FEMA-mapped or County Ordinance No. 458 floodplain
Projects with potential maintenance difficulties due to access, right of way constraints
Any multi-use features within rights of way intended for District operation and maintenance

Plan preparation for this submittal should meet the requirements of the District's Drafting Manual for 30% plans.

For typical storm drain and hardened channels that don't include the above type feature, the first submittals are to be as described below.

2.8.1.1 Catch Basin Hydrology and Preliminary Design

Prepare the hydrology study for and preliminary sizing of proposed inlets/catch basins.

- The 10-year and 100-year Rational Method hydrology calculations shall be completed, together with street flow capacities to determine proposed inlet/catch basin locations.
- Catch basin lengths and locations shall be determined and compared to design topographic mapping to identify any space constraints. Ponding depths and flow-by rates shall be determined with flow-by values being added to the next downstream catch basin depending upon street carrying capacity.
- Identify preliminary horizontal alignments for connector pipes, including estimated sizes. These horizontal alignments will be used to help identify if/where any potential utility conflicts may exist (for utility coordination) and where junctions with the mainline will be located.

2.8.1.2 Mainline Alignment and Profile

For the preferred alignment, prepare initial engineering horizontal alignment and profile plots for the complete length of the project. Project mainline and lateral alignments shall be plotted on design mapping to scale with offsets to roadway centerline and property lines dimensioned. Profile shall show enough detail to allow an understanding of how the project will accommodate all vertical constraints, major utilities, and outlet conditions.

Prepare and overlay the HGL (pressurized) or Water Surface (non-pressurized) onto the profile. The hydraulics for the mainline/laterals should utilize the mainline/lateral flow rates identified by Planning during the Preliminary Design Report (PDR) and the junction locations and estimated sizes determined by the Catch Basin Hydrology and preliminary design. Confirm all boundary condition assumptions (U/S and D/S controlling water surfaces, etc.).

The Design Engineer shall determine if right of way (ROW) will be required in order to construct the project. If ROW is required, the Design Engineer shall determine the limits of land required for the project as well as ROW needed for mitigation and downstream impacts. The Design Engineer will also need to identify if the ROW is fee, easement, temporary construction easement (TCE), etc.

2.8.2 Plan Check 1 – Engineering Plans and Studies

The first submittal (unless a preliminary design submittal is required) is a 60% to 90% plan level, submitted in digital format (CD or USB, or link to digital files), and include:

- Plan Check Application Form
- Plan Check Deposit Based Fee Worksheet
- A copy of the Conditions of Approval (COA) with discussion as necessary to explain any unusual means of satisfying the COA or any contemplated deviations
- Drainage Improvement plans prepared consistent with the District's Drafting Manual including all elements and sheets required for both 30% and 60% plans
- Drainage report consisting of a hydrology and hydraulics study as outlined in Section 2.3.
- Identification of all necessary rights of way and easements, clearly labeled on the plans
- Scour analysis/calculations, as appropriate
- Debris potential and burned condition assessment, as appropriate
- Conditional Letter of Map Revision/Letter of Map Revision (CLOMR/LOMR) documents, when applicable
- Final geotechnical investigation, including infiltration testing, as necessary
- The following items as applicable: street improvement plans, rough grading and erosion control plans, final tract/parcel map, Environmental Constraint Sheet (ECS), sewer and water plans
- Identification of any multi-use facilities together with all appurtenances (e.g., landscaping) and types of surfaces required
- Basin Survey Control Sheet as part of storm drain improvement plan set, as applicable
- Maintenance Exhibit to be incorporated in the storm drain improvement plan set per Drafting Manual criteria

2.8.3 Plan Check 1 – Environmental Documents

The first submittal shall include the following environmental documents pertaining to the construction, operation, and maintenance of any facilities to be owned and/or operated by the District. These documents shall be submitted in digital format:

- CEQA documents (i.e., Initial Study, MND, EIR, etc.)
- MSHCP compliance documents, if applicable

- Regulatory permits (i.e., 401, 404, 1602, and related habitat assessments, survey reports, Biological Opinion, Incidental Take) and permit application packages
- Written confirmation from applicable agencies where permits are deemed unnecessary
- Final and permitted maintenance plans, if any
- Mitigation and monitoring plans, if any
- Set of relevant District facility improvement plans

2.8.4 Plan Check 2 – Engineering Plans and Studies, and Environmental Documents

The second submittal is to be prepared to a 90% to 100% level. Plans and/or studies may be submitted in digital format with prior agreement/direction from the plan checker. This submittal shall include:

- Plan check response letter addressing each comment from prior review. Comments not addressed in prior review will result in an additional plan check.
- Prior submittal redlines
- Revised plans, studies, and maps addressing prior review comments, prepared consistent with the District's Drafting Manual including all elements and sheets required through 90% as applicable.
- Supplementary materials requested in prior review comments
- Identification and discussion of any phasing and of any temporary measures or interim facilities required to support phased construction
- D-Load calculations (where RCP is used)
- Structural calculations as needed for any non-standard design elements
- Letters of permission from affected property owners for offsite grading, ponding, or discharge of concentrated drainage
- Water Quality Management Plan for projects exceeding Priority Development Project thresholds (if required by plan checker)

2.8.5 Plan Check 3 – Engineering Plans and Studies, and Environmental Documents

The third submittal shall be prepared to a 100% level. Plans and/or studies may be submitted in digital format with prior agreement /direction from the plan checker. This submittal shall include:

- Plan check response letter addressing each comment from prior review. Comments not addressed in prior review will result in an additional plan check.
- Prior submittal redlines
- Revised plans, studies, and maps addressing prior review comments prepared consistent with the District's Drafting Manual including all elements and sheets required through 90% as applicable.
- Supplementary materials requested in prior review comments
- Final Map and applicable right of way documents (legal descriptions, plats, closure reports)
- Cooperative Agreement application package with all attachments, available on District website or by contacting Contract Services Section. Please note that if there are any outstanding project issues that could result in changes to the alignment, facility size, rights of

way, and/or maintenance responsibilities, submittal of the Cooperative Agreement application package should be deferred until these issues have been fully resolved.

- The following items may be requested as early as the third submittal. Submit these only when requested by the plan checker:
  - Reduced sized (11"x17") storm drain and/or channel improvement plans – 10 sets
  - Construction materials quantity and cost estimate – to be used in preparation of the bonding letter to be prepared by the plan checker

2.8.6 Plan Check 4 and Beyond – Engineering Plans and Studies, and Environmental Documents

Submittals four and beyond, if necessary, shall be prepared to a 100% level as previously described. Plans and/or studies may be submitted in digital format with prior agreement direction from the plan checker. The plan checker may request a pre-submittal meeting to ensure unresolved issues are being adequately addressed. Plan Check 4 and subsequent submittals shall include:

- Plan check response letter addressing each comment from prior review. Comments not addressed in prior review will result in an additional plan check.
- Prior submittal redlines
- Revised plans, studies, and maps addressing prior review comments
- Supplementary materials requested in prior review comments
- Any documents not previously submitted as outlined above
- As requested by the plan checker:
  - Reduced sized (11"x17") storm drain and/or channel improvement plans – 10 sets
  - Construction materials quantity and cost estimate - to be used in preparation of the bonding letter to be prepared by the plan checker

2.8.7 Mylars and Record Drawing (Physical or Digital)

Mylars and Record Drawings shall be prepared, signed, and stamped by the design engineer. If digital (preferred), it shall be signed and submitted in conformance to the Digital Record Drawing Creation and Revision Procedure with Electronic Signatures and submitted for District signature when requested by the plan checker. For projects located within an incorporated city, the mylars shall be signed by the City Engineer (or another authorized City signatory) prior to submittal to the District. Note that the District will not sign mylars until the Cooperative Agreement is executed.

2.9 MAINTENANCE REQUIREMENTS

Maintenance access shall be analyzed using vehicle turning analysis software as determined by the District's Operations Division to ensure large equipment have sufficient clearance for operations. Please reference Section 10 for minimum ingress/egress requirements.

2.10 COST ESTIMATE

An initial materials quantity and cost estimate for construction of the facility shall be submitted with the first plan check for use in computing plan check fees. On or about Plan Check No. 3, an updated construction materials quantity and cost estimate based on the District's Deposit Based Fee

Worksheet will be requested by the plan checker for the purpose of preparing a bonding estimate letter.

2.11 PLANS/DRAFTING STANDARDS

Construction drawings of District facilities shall be prepared on plan sheets (typically 22"x34") to allow a 50% reduction to 11"x17" for reproduction, microfiche, and filing. Sizes other than these require Agency approval. Upon request of the plan checker, a digital pdf copy of the final plans and supporting studies shall also be provided for archiving purposes. All plan sheets and drafting shall conform to this manual and the latest edition of the District Drafting Manual.

2.12 DEVELOPMENT AGREEMENT

A Cooperative Agreement is required for all developer/partner storm drain facilities to be maintained by the District. The Cooperative Agreement application package with all attachments is available on the District website (https://rcflood.org/development-process) or by contacting the District's Contract Services Section. Please note that if there are any outstanding project issues that could result in changes to the alignment, facility size, rights of way, and/or maintenance responsibilities, submittal of the Cooperative Agreement application package should be deferred until these issues have been fully resolved.

2.13 REFERENCES

Riverside County, March 2020, Memorandum of Understanding Riverside County on Behalf of its Transportation Department and Riverside County Flood Control and Water Conservation District for Design, Construction, Inspection and Maintenance of Flood Control Drainage Facilities. rcflood.org>Business>Engineering Tools>MOU District Transportation

Riverside County Flood Control and Water Conservation District Drafting Manual , August 2018. rcflood.org>Business>Engineering Tools>Drafting Manual

3 HYDROLOGY

3.1 METHODOLOGY

The determination of flood hydrology for designing stormwater facilities is to be made using one of the following:

- With prior approval of the District, existing studies of record may be utilized. Such studies may include FEMA Flood Insurance Studies, Master Drainage Plans, and approved design reports for adjacent facilities.
- If appropriate existing information is not available, then the procedures set forth in the Riverside County Flood Control and Water Conservation District Hydrology Manual (RCFCWCD, 1978), herein after referred to as the Hydrology Manual, are to be used.

Peak flow rates and volumes from studies or reports of record being utilized for final design must be checked for reasonableness. Parameters and assumptions (such as land use, rainfall data, tributary area, etc.) from these studies must be confirmed to ensure flow rates and previously determined facility sizes are still applicable. Analysis of interim conditions must also be considered when this condition governs for design over the ultimate condition.

The selection of the procedure used to determine design flood hydrology is dependent upon the intended application. For small watersheds (defined as less than 300 to 500 acres), the use of the Rational Method is stipulated. Use of the Rational Method will only produce peak discharges and should not be used if a runoff hydrograph is needed, such as for the routing of flow through a detention facility. For larger drainage networks or basin routing analysis, a Synthetic Unit Hydrograph rainfall-runoff model is required. The Hydrology Manual provides guidance in the development of such a model and the estimation of the necessary input parameters to the model.

Although not necessarily required, the U.S. Army Corps of Engineers' Hydrologic Modeling System HEC-HMS can be used in preparing Synthetic Unit Hydrograph Method calculations as outlined in the Hydrology Manual. The District website includes a link to a HEC-HMS preprocessor specific to Riverside County. The preprocessor calculates the effective rainfall, lag time, and S-graph ordinates necessary for preparing a Synthetic Unit Hydrograph model in HEC-HMS that is consistent with District methods.

Diversion of drainage is not allowed (i.e., all water entering a storm drain system must be discharged within the same drainage basin or watershed that it was collected in).

For multi-use detention facilities, additional hydrologic analyses may be required to meet pollutant control, hydromodification, and Riverside County Increased Runoff Criteria (Section 10.4). For pollutant control and hydromodification performance standards, the Design Engineer is directed to the Water Quality Management Plan (WQMP) for the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable. Multi-use detention basin storage volume guidelines are provided in Section 10.5 CONJUNCTIVE/MULTI -USE BASIN FACILITY .

3.2 CRITERIA

The Hydrology Manual shall be used to develop the design discharge for storms of frequencies up to and including the 100-year event.

3.3 DRAINAGE PLANNING

Drainage planning can be encountered on both basin-wide (e.g., regional detention basins, Master Drainage Plan (MDP) channels and storm drains) and local scales (e.g., development project onsite storm drain and increased runoff basins). Drainage planning for development projects should be done in the earliest stages of the planning process. A development project drainage plan shall include a hydrologic analysis for onsite and offsite runoff, a preliminary hydraulic analysis for proposed facilities, and outline a recommended plan for handling the runoff.

When evaluating a basin-wide plan (i.e., Master Drainage Plans, Specific Plans for large developments), the designer must evaluate alternatives to find the most cost-effective overall solution for the general public. When preparing drainage plans on a local scale (i.e., development projects), the designer shall demonstrate conformance with basin-wide drainage plans where they exist or shall demonstrate that the plan will not increase the cost of providing basin-wide drainage for the local governing agency or the District.

When preparing or referencing master drainage plans, it is important to recognize that these documents provide only planning level facility sizes, geometries, and alignments, and that each of these facets must be further examined during the final design phase. Sizing of riprap, structural design, and selection and detailing of peripheral elements (e.g., inlets, trash racks, fencing, etc.) are typically excluded from Master Drainage Plans and are completed in later phases using the criteria outlined in this manual. While Master Drainage Plans may consider anticipated right of way acquisition costs, the mere fact that facility is shown on an MDP does not guarantee that the right of way required to build the facility along that alignment can be acquired. Furthermore, Master Drainage Plans reflect facility sizes anticipated under ultimate build-out of the plan per County’s General Plan, whereas individual facilities may have to be built larger than that reflected in the Master Drainage Plan should interim condition flow rates dictate.

Preparation of a development project drainage plan typically begins with analysis of the offsite runoff impacting the project site, anticipated onsite generated runoff, and a conceptual layout of the drainage system required to accommodate these flows. Drainage entrance and exit points to a development site must typically remain in the original location unless permission to alter drainage patterns is granted by owners of impacted properties.

Development projects must be protected from flooding in both the interim and ultimate conditions. It is important to recognize during the project planning phase that in some cases interim conditions may require drainage facilities be designed to accommodate larger peak flow rates than would be required for the ultimate condition. Consideration for facilities to accommodate both interim and ultimate condition flow rates is required for all projects.

3.3.1 Compatibility With Existing Systems

In some cases, proposed developed condition (both interim and ultimate condition) flow rates must be mitigated to pre-developed condition flow rates at the downstream project boundaries if existing downstream storm drain conveyance is inadequate for the increased flow rates – this can be done using combinations of low impact development techniques (see Section 3.5) and detention basins (see Section 10 DETENTION BASINS). Whereas, in other cases, where presence of an adequate outlet has been acknowledged by the District, flow rates at project discharge points may not need to be mitigated to the pre-developed condition flowrates. Determination of discharge mitigation requirements by the District is typically made during the project entitlement process.

3.4 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.

Section F of the District Hydrology Manual covers the peak rate bulking factor. The bulking factor and associated methodology will be updated as part of the Hydrology Manual revision.

In lieu of using the simplified method presented in Section F of the District Hydrology Manual, a bulking factor study can be conducted by a California Registered Engineer and submitted to RCFCD for review and approval.

3.5 REFERENCES

Riverside County Flood Control and Water Conservation District Hydrology Manual , April 1978. rcflood.org>Business>Engineering Tools>RCFC Hydrology Manual

Riverside County Flood Control and Water Conservation District Design Handbook for Low Impact Development Best Management Practices, September 2011. https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/

Riverside County Flood Control and Water Conservation District Santa Margarita River Watershed Region Design Handbook for Low Impact Development Best Management Practices, June 2018. https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/

Riverside County Flood Control and Water Conservation District Whitewater River Watershed Region Stormwater Quality Best Management Practice Design Handbook for Low Impact Development, June 2014. https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/

U.S. Army Corps of Engineers (USACE), 2018, *Hydrologic Modeling System HEC-HMS, User's Manual*.

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4 HYDRAULICS OF GRAVITY FLOW SYSTEMS

4.1 FUNDAMENTALS OF FLOW IN STORMWATER DRAINAGE

4.1.1 Flow Classification

Open channel flow is classified in various ways based upon how the flow varies spatially and temporally. Classification types of open channel flow include steady and unsteady flow, uniform and varied flow.

4.1.1.1 Steady Flow

Steady flow is one in which all conditions at a particular point in a stream remain constant with respect to time (Daugherty and Franzini, 1977). Most commonly and for the purposes of this manual, a channel experiencing steady flow is more simply defined as a one that has a constant flow rate at a given point in a channel for the time period under consideration.

Steady Flow = Constant Flow Rate over Time

4.1.1.2 Unsteady Flow

Flow is unsteady if the flow conditions such as flow rate and depth change over time. All storm runoff is inherently unsteady flow, however, open channels and storm drains are commonly designed considering steady flow conditions at a given time step of the hydrograph, typically the peak flow rate. However, in some cases such as large natural systems, unsteady flow may be considered as a storm hydrograph flows through the system.

Unsteady Flow = Variable Flow Rate over Time

4.1.1.3 Uniform/Normal Flow

Changes in flow conditions along the length of the channel is the criterion used in the determination of uniform and varied flow. A truly uniform flow is one in which the velocity is the same in both magnitude and direction at a given instant at every point in the fluid (Daugherty and Franzini, 1977). Most commonly and for the purposes of this manual, open channel flow is considered uniform if the flow depth is the same at every point along the length of a channel that has uniform geometry, slope, and material. Uniform flow is also called normal flow, and the flow depth under uniform flow conditions is referred to as normal depth. Refer to Section 4.2.6.1 for more detailed information in regard to the computation of normal depth.

Uniform Flow = Constant depth over a length of uniform channel

For a given channel geometry, roughness, discharge, and slope, there is only one possible depth for maintaining uniform flow. This depth is referred to as the "normal depth." For uniform flow the water surface profile will be parallel to the channel bottom profile. While uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, a uniform-flow approximation can be adequate for early planning purposes, and some limited design purposes where allowed in this manual.

4.1.1.4 Varied Flow

Channel varied flow conditions occur when hydraulic conditions such as depth, change along the length of the channel. The change in depth occurs either rapidly or gradually depending upon the channel geometry and flow constraints. The flow is **rapidly varied** if the depth changes abruptly over a relatively short distance. Examples of rapidly varied flow include local phenomena, such as hydraulic jumps and hydraulic drops, typically where flow transitions from supercritical to subcritical. Under steady flow conditions, if the depth of flow along the length of the channel gradually and smoothly increases or decreases it is **gradually varied**. This is the usual condition in open channel flow. Gradually varied flow occurs under either subcritical or supercritical flow regimes. Water surface profile computations are required to estimate the depth of flow for gradually and rapidly varied flow conditions at any given location as described in Section 4.4.

Varied Flow = Changing depth over a length of channel

4.1.2 Energy States / Flow Regimes

To perform hydraulic calculations, you must first understand what energy state the water is flowing in. Depending on the level of energy, flow will occur in one of three states:

- Critical
- Subcritical
- Supercritical

The energy state for a particular storm drain or channel system can be determined by plotting the depth of flow on Specific Energy curves or determining the Froude Number as defined below.

4.1.2.1 Specific Energy

Specific energy in a channel section is defined as the energy per pound of water at any section of a channel measured with respect to the channel bottom and may be expressed as:

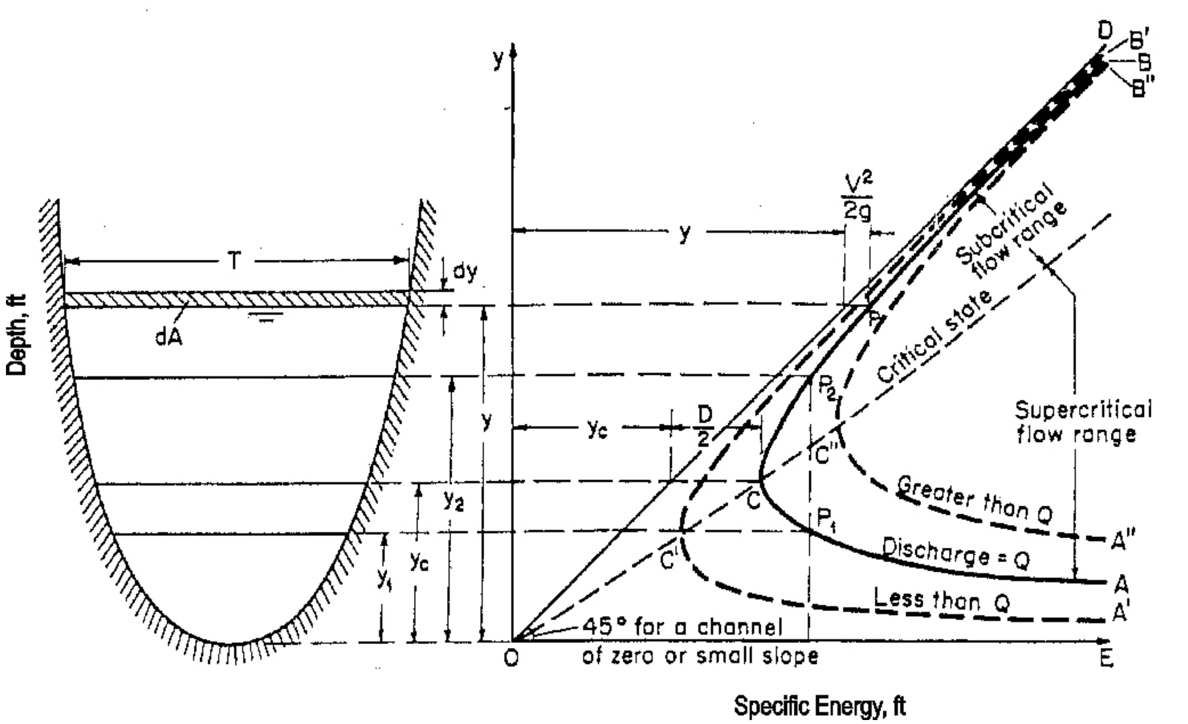
$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2} \tag{4.1}$$

When the depth of flow is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained (Figure 4.1). The specific energy curve has two limbs, AC and BC. The limb AC approaches the horizontal axis asymptotically toward the right. The limb BC approaches the line OD as it extends upwards and to the right. The line OD has an angle of inclination equal to 45^o. At any point P on this curve, the ordinate represents the depth of flow, and the abscissa represents the specific energy that is equal to the sum of the pressure head, y, and the velocity head, $V^2 / 2g$. The curve shows that, for a given specific energy, there are two possible depths, the low stage, y_1, and the high stage, y_2. The low stage is called the alternate depth of the high stage and vice versa.

At point C, the specific energy is a minimum and the stage is at critical depth. When the depth of flow is greater than the critical depth, the velocity of flow is less than the critical velocity and the flow is subcritical. When the depth of flow is less than the critical depth, the flow is supercritical. Inspection of the energy curve in the vicinity of critical depth reveals that a small change in the energy will result in a relatively large change in the depth of flow. For this reason, it is strongly recommended that flow depths producing Froude numbers between 0.87 and 1.13 inclusive be avoided.

Figure 4.1: Specific Energy Curve

(MODIFIED FROM: Chow, 1959)



4.1.2.2 Froude Number

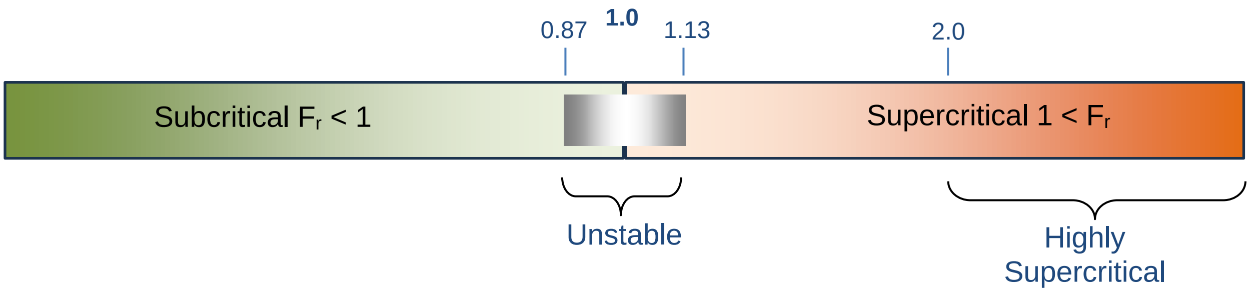
The energy state of open channel flow is governed by the effects of viscosity and gravity relative to the inertial forces of the flow. The effect of gravity on the state of flow is represented by a ratio of inertial forces to gravity forces. This ratio is given by the Froude Number (dimensionless), defined as:

$$F_r = \frac{inertial\ forces}{gravitational\ forces} = \frac{V}{\sqrt{gD_h}} \tag{4.2}$$

where:

V	=	mean velocity (ft/sec)
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
D_h	=	hydraulic depth (ft) = A/T
A	=	cross-sectional area of the water (sq ft)
T	=	width of the free surface (ft)

When F_r is equal to 1, the flow is considered critical flow, and the corresponding depth is referred to as critical depth. This flow condition is unstable and flow depths at or near critical depth should be avoided. If F_r is less than 1, the flow is subcritical and gravity forces dominate. When F_r is greater than 1, the flow is supercritical and inertial forces predominate.



Steep Channels or Highly Non-uniform Flow

If the slope is large and/or the flow velocity is highly non-uniform within a cross section, the Froude number should be calculated using the following form:

$$F_r = \frac{V}{\sqrt{gD_h \cos\theta/\alpha}} \tag{4.3}$$

where:

V	=	mean velocity (ft/sec)
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
D_h	=	hydraulic depth (ft) = A/T
A	=	cross-sectional area of the water (sq ft)
T	=	width of the free surface (ft)
θ	=	the bed slope (degrees)
α	=	velocity head coefficient, which corrects for the non-uniformity of the velocity in the channel (see Equation (4.10) in Section 4.2.3 (ft)

4.1.2.3 *Critical Flow*

The critical state of flow through a channel is characterized by several important conditions regarding the relationship between the flow, specific energy, and slope of a particular hydraulic cross section (Figure 4.1). Critical state is characterized by the following conditions:

- For a given flow rate (Q), the specific energy ($E = y + v^2/2g$) is at a minimum.
- The discharge (Q) is at a maximum for a given specific energy (E).
- The specific force is a minimum for a given discharge (Q). See Section 4.2.5 for definition of specific force.
- The Froude Number (Fr) is 1.0.

Typically, channels should not be designed to flow at or near critical state ($0.87 \leq F_R \leq 1.13$). If the critical state of flow exists throughout an entire reach, the channel flow is critical, and the channel slope is at *critical slope* (S_c). A slope less than S_c will cause subcritical flow. A slope greater than S_c will cause supercritical flow.

4.1.2.4 *Critical Depth*

By substituting $V^2 = Q^2A^2$ into Equation (4.2) for a Froude Number Fr = 1.0, and rearranging terms, we can obtain a general expression for critical flow that is applicable to any channel cross section:

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{4.4}$$

Substituting values for a rectangular channel configuration [$T=b$; $A=(b)(y_c)$] into Equation (4.4), then rearranging, the critical depth for a rectangular channel can be determined as indicated in Equation (4.6). See Table 4.5 for resources regarding additional conduit shapes.

$$\frac{Q^2}{g} = \frac{[(b)(y_c)]^3}{b} \tag{4.4}$$

$$y_c = \sqrt[3]{\frac{Q^2}{gb^2}} \quad \text{(rectangular channel)} \tag{4.5}$$

where:

yc = critical depth (ft)
Q = flowrate (cfs)
g = acceleration due to gravity 32.2 ft/sec ^{2}
b = base width of rectangular channel (ft)

EXAMPLE 4.1: What is the critical depth of flow for 400 cfs flowing in a rectangular channel 10.0 feet wide?

$$y_c = \sqrt[3]{\frac{Q^2}{gb^2}} \tag{4.5}$$

$$y_c = \sqrt[3]{\frac{400^2}{(32.2)\ 10^2}}$$

$$y_c = 3.68\ ft$$

4.1.2.5 Subcritical Flow

Flows producing Froude numbers less than 1.0 are subcritical and have the following general characteristics relative to critical depth:

- Slower velocities
- Greater depths
- Lower hydraulic losses
- Less erosive power
- Less sediment carrying capacity
- Behavior easily described by relatively simple mathematical equations
- Surface waves propagate upstream

4.1.2.6 Supercritical Flow

Flows with Froude numbers greater than 1.0 are supercritical and have the following general characteristics relative to critical depth:

- Higher velocities
- Shallower depths
- Higher hydraulic losses
- More erosive power
- More sediment carrying capacity
- With higher Froude numbers (highly supercritical), behavior becomes less predictable mathematically
- Surface waves propagate downstream only

4.2 EQUATIONS OF FLOW

4.2.1 Assumptions and Limitations

General Flow Equation - For any flow, the discharge, Q, at a channel section is expressed by:

$$Q = AV \tag{4.6}$$

where:

Q = flow rate (cubic feet per second)
A = cross section area of the flow measured normal to the direction of flow (sq ft)
V = mean velocity (ft/sec)

1-Dimensional Flow – This manual presents hydraulic analysis methods that are applicable to systems that can be assumed to operate as one-dimensional flow, or where all flow and velocity is heading in one direction, along the length of the drainage system being analyzed. Scenarios where flow will spread out in multiple directions are more appropriate for 2-dimensional analyses that are beyond the scope of this manual.

Physical Laws governing flow – The calculations presented in this manual are governed by the following underlying principles:

- Conservation of Mass (Continuity)
- Conservation of Energy and Momentum

Slope – The equations as presented are generally valid for slopes under 10%. For larger slopes, additional factors must be considered such as air entrainment, and the effects of slope on the normal and axial components of depth, weight, and momentum.

Steady Flow – While all storm runoff is inherently unsteady flow, for open channels and storm drains the methods presented in this manual assume steady flow conditions (constant Q) that correspond to a given time step of the hydrograph, typically the peak flow rate. Where junctions in the stormwater drainage system add flow (thus changing the Q), the junctions are analyzed using the methods described in Section 4.3.2, and then the new steady-state flow rate (with the changed Q) is analyzed upstream and downstream accordingly.

Uniform vs Varied Flow – True Uniform flow is rare in stormwater drainage. Most stormwater drainage systems, including all storm drains and channels that are identified in a District Master Drainage Plan or that will be submitted to the District for operation and maintenance, should be designed to determine where Varied Flow and Uniform Flow exist, and to plot an accurate water surface profile or hydraulic grade line, as applicable.

4.2.2 Conservation of Mass (Continuity)

Under steady flow conditions, 1-dimensional flow conditions, the flow rate is constant and therefore:

$$Q = A_1 V_1 = A_2 V_2 \tag{4.7}$$

The subscripts denote adjacent channel sections. Equation (4.7) is known as the Continuity Equation.

4.2.3 Conservation of Energy

The First Law of Thermodynamics states that energy can neither be created nor destroyed; it can only be transformed. Thus, in the case of an open channel carrying a steady flow, the total energy at any two points must be equal, less any energy transformation (losses) that occur between those two points.

General Energy Formula: At a given cross section, the total energy at any point (E_T) is the sum of kinetic energy (Equation (4.8)) and potential energy at that point.

$$E_T = \textit{Kinetic Energy} + \textit{Potential Energy}$$

$$E_T = \alpha \frac{V^2}{2g} + y + z \tag{4.8}$$

where:

α	=	velocity head coefficient; kinetic energy coefficient
V	=	average velocity (fps)
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
y	=	depth of flow (ft)
Z	=	elevation of channel bottom (ft)

The velocity head coefficient, α, is a correction to account for the non-uniformity of the velocity in the channel. For channels of regular cross section and fairly straight alignment, the effect of non-uniform velocity distribution on the computed velocity head is small, especially when compared to other uncertainties involved in the computation. Therefore, α is often assumed to be 1.0. The District allows using α = 1.0 for typical engineered open channel flow conditions if they have a simple prismatic cross section. For natural or non-prismoidal channels, calculation of the coefficient may be required.

The general form of the coefficient equation is provided below, however, determining the flow rates, velocities, and areas of flow in the sub-portions of the channel cross sections requires more advanced analyses that are beyond the scope of this manual.

$$\alpha = \frac{[Q_i V_{1i}^2 + Q_{ii} V_{ii}^2 + \cdots + Q_n V_n^2]}{Q \bar{V}^2} = \frac{\sum v^3 A}{\bar{v}^3 A} \tag{4.9}$$

where:

α	=	velocity head coefficient
$\bar{v}$	=	average velocity of the cross section
Vn	=	velocity in the ‘n’ portion of the channel cross section
Qn	=	flow rate in the ‘n’ portion of the channel cross section

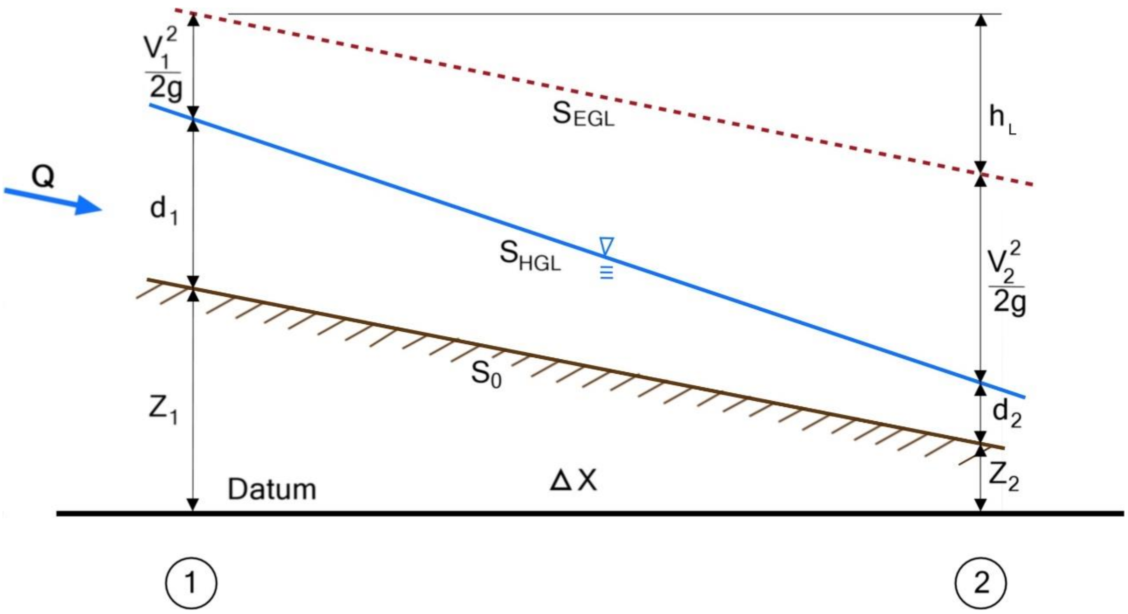
Application: Equation (4.9) for the conservation of energy is useful in situations where the energy losses between two points (h_L) can be directly calculated (estimated) using established methods. In the case of gradually varied flow (see Section 4.4.3.2). the principal source of energy losses are due to friction, which can be estimated using Manning’s equation (see Section 4.2.6). In the case of Rapidly Varied Flow such as a hydraulic jump, there are no methods to directly calculate all of the losses h_L, so the principle of conservation of Momentum is instead used.

Conservation of Energy: Applying Equation (4.9) to open channel flow, the conservation of energy results in the following relationship:

$$E_1 = E_2 + h_L$$

Where, h_L is the energy lost between the two points in the channel.

Figure 4.2: Energy in Gradually Varied Open Channel Flow
(modified Chow, 1959)



Equation (4.10) below shows the various components of the energy equation applied to gradually varied open channel conditions:

$$E_T = \alpha_1 \frac{V_1^2}{2g} + y_1 + z_1 = \alpha_2 \frac{V_2^2}{2g} + y_2 + z_2 + h_L \tag{4.10}$$

where:

α	=	velocity head coefficient; kinetic energy coefficient
V_1	=	velocity at section 1 (fps)
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
y_1	=	depth of flow at section 1 (ft) = d_1 cos θ
Z_1	=	elevation of channel bottom at section 1 (ft)
V_2	=	velocity at section 2 (fps)
Y_2	=	depth of flow at section 2 (ft) = d_2 cos θ
Z_2	=	elevation of channel bottom at section 2 (ft)
h_L	=	energy losses between section 1 and 2 (See Section 4.3)

4.2.4 Conservation of Momentum

Momentum Formula: The general formula for momentum (M) is:

$$M = \dot{mass} \times velocity$$

Where, similar to how $velocity$ is the distance per unit time, $\dot{mass}$ is the mass flow rate, or the mass per unit time. For hydraulics, the mass term can be described as follows:

$$\dot{mass} = \frac{mass}{time} = \frac{\rho \cdot Volume}{time} = \left(\frac{\gamma}{g}\right) \times Q \tag{4.11}$$

where:

γ = Unit weight of water = 62.4 pounds per cubic foot
Q =flow rate (cubic feet per second)
g= acceleration due to gravity 32.2 ft/sec ^{2}

Therefore, the formula for momentum can be re-written as follows:

$$M = [m\dot{a}ss] \times [velocity] = \beta \cdot \left[\left(\frac{\gamma}{g}\right) \times Q\right] \times [V] \tag{4.12}$$

Or by substituting $V = \frac{Q}{A}$

$$M = \beta \frac{\gamma Q^2}{gA} \tag{4.13}$$

where:

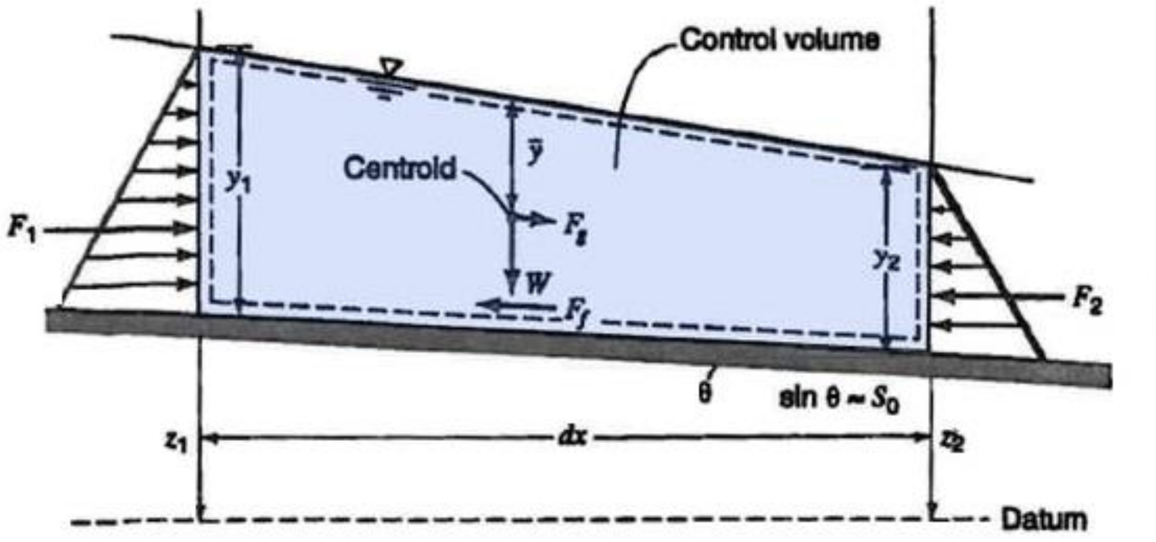
M= Momentum
β = Momentum correction factor (see Equation (4.15) below)
γ = Unit weight of water = 62.4 pounds per cubic foot
Q =flow rate (cubic feet per second)
A = Cross sectional area of flow (square feet)
g = acceleration due to gravity 32.2 ft/sec ^{2}

The momentum correction factor β accounts for non-uniform velocity distribution that may exist when the channel cross section is natural or non-prismatic. For prismatic engineered channels β is commonly assumed to be equal to 1.0. For other cases, β is calculated as shown below, however, determining the flow rates, velocities, and areas of flow in the sub-portions of the channel cross sections requires more advanced analyses that are beyond the scope of this manual.

$$\beta = \frac{\sum v^2 A}{\bar{v}^2 A} \tag{4.14}$$

Conservation of Momentum: The principle of Conservation of Momentum is that in the absence of any forces acting on a body of water, momentum between two points would be equal. Where such forces do exist, you can equate the forces acting on that body to a change in momentum, following the equation below:

Figure 4.3 External Forces Acting on the Boundary of the Control Volume



$$M_2 - M_1 = \sum F_x \tag{4.15}$$

Where M_1 and M_2 are the vector momentum in the x direction.

The typical forces acting on the body of water between points 1 and 2 consist of:

- F_{P1}: Force due to hydrostatic pressure at point 1
- F_{P2}: Force due to hydrostatic pressure at point 2
- F_R: Retardation force, typically due to friction (see Section 4.3.1)
- F_i: axial horizontal component of hydrostatic pressure force on the invert (based on channel slope)
- F_w: axial horizontal component of hydrostatic pressure force on the walls (for transitions)

$$M_2 - M_1 = [F_{P1} - F_{P2} - F_R - F_{Pi} \pm F_{Pw}] \tag{4.15}$$

Note: the sign of the forces $F_{(x)}$ depends on the direction it is acting. The positive direction is for forces that are acting in the downstream direction.

The above formula can be re-written into the common 'Pressure + Momentum' form:

$$[F_{P2} + M_2] = [F_{P1} + M_1] - F_R - F_{Pi} \pm F_{Pw} \tag{4.16}$$

Or other forms such as (for constant Q):

$$\gamma A_1 Y_{CG1} + \frac{\gamma Q^2}{g A_1} = \gamma A_2 Y_{CG2} + \frac{\gamma Q^2}{g A_2} - F_R - F_{Pi} \pm F_{Pw} \tag{4.17}$$

$$\frac{\gamma}{g} \cdot Q \Delta V = [F_{P1} - F_{P2} - F_R - F_{Pi} \pm F_{Pw}] \tag{4.18}$$

For a short horizontal reach of constant prismatic channel, the external force of the friction and the weight effect of water can be ignored, further simplifying this equation to:

$$\frac{\gamma}{g} \cdot Q\Delta V = [F_{P1} - F_{P2}] \tag{4.18}$$

The forces $F_{(x)}$ can be calculated using the following formulas:

$F_{P1,P2}$ = Pressure x Area

$$F_{P1} = \gamma \bar{z}_1 \cdot A_1 \tag{4.19}$$
$$F_{P2} = \gamma \bar{z}_2 \cdot A_2 \tag{4.19}$$

where:

γ = unit weight of water

$\bar{z}$ = depth to the centroid of the respective water areas, measured from the surface of flow

A = cross sectional area of water. In the case of Fi and Fw, the area is the cross-sectional projection of the area occupied by the change in invert and walls, relative to the reference section

$F_{Pi,Pw}$ = $\text{Pressure}_{\text{average}}$ x $\text{Area}_{\text{projected}}$

$$F_{Pi,Pw} = \gamma \bar{z}_{avg} \cdot A_{projected} \tag{4.20}$$

where:

γ = unit weight of water

$\bar{z}_{avg}$ = depth to the centroid of the three-dimensional prism of water occupied by the change in invert elevation and cross-sectional geometry between points 1 and 2 , measured from the surface of flow

$A_{projected}$= cross sectional area of water. In the case of Fi and Fw, the area is the cross-sectional projection of the area occupied by the change in invert and walls, relative to the reference section

Formulas for common cases of F_{Pi}, F_{Pw} can be found beginning on page F-6 of the Los Angeles County Hydraulics Manual (1982) (Referred to simply as P_i and P_w in that manual)

F_R in most cases will be equal to the force of resistance due to friction, F_{Pf}.

F_{Pf} = [Weight of Volume of Water] x [Average Friction Slope]

$$F_{Pf} = [\gamma \cdot L \cdot A_{avg}] \times \frac{S_{f1} + S_{f2}}{2} \tag{4.21}$$

where:

γ = unit weight of water

L = Length between sections 1 and 2 (feet)

A_{avg} = Average of the cross-sectional area of water at section 1 and 2.

S_f = Friction slope calculated using Manning's equation, per Section 4.2.6.

Formulas for common cases of F_{Pf} can be found beginning on page F-6 of the Los Angeles County Hydraulics Manual (1982) (Referred to simply as P_f in that manual)

Application: Equations (4.14) through (4.21) for the conservation of momentum is useful in situations where the total energy losses between two points cannot be directly calculated, such as in Rapidly Varied Flow, or the Hydraulic Analysis of junctions. Junctions add a third series of terms for the lateral flow entering the system, as shown in Section 4.3.2.

4.2.5 Specific Force

The specific force, F, in a channel is the sum of the pressure force + momentum, per unit weight of water, and may be expressed by the general function:

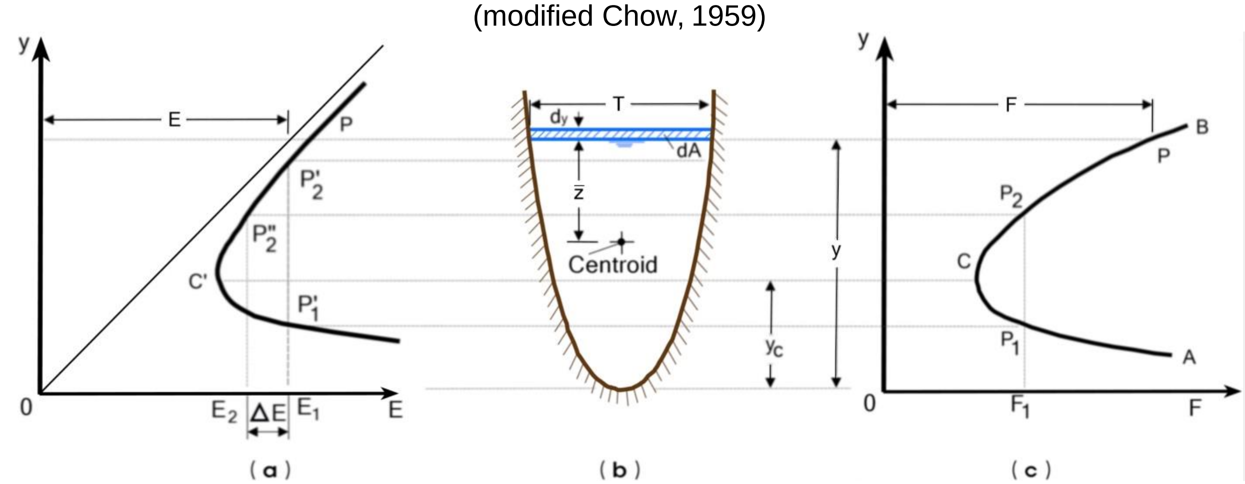
$$F = \frac{[F_P + M]}{\Upsilon} = Q^2/gA + \bar{z}A \tag{4.22}$$

where:

Q = flow rate (cfs)
g = acceleration due to gravity 32.2 ft/sec ^{2}
A = cross sectional area of flow (square feet)
$\bar{z}$ = depth from the water surface to the centroid of A (feet)

For a given channel section and flow rate, a graph of the specific force versus depth can be created, referred to as the specific force diagram. Two possible depths are evident for a given value of the specific force. These depths are referred to as the initial and sequent (or conjugate) depths. At the point of minimum specific force the depth is equal to critical depth. Figure 4.4 shows a comparison of the specific energy and specific force diagrams.

Figure 4.4: Specific-Force Curves Supplemented with Specific-Energy Curves
(a) Specific-Energy Curve; (b) Channel Section; (c) Specific-Force Curve.
(modified Chow, 1959)



4.2.6 Manning's Equation

Manning's equation is one of the most widely accepted and commonly used equations for determining the hydraulic capacity of channels and storm drains based on the frictional energy losses that occur as stormwater flows through the system.

4.2.6.1 General Form

Manning's Equation is as follows:

$$V = \frac{1.486}{n} R^{2/3} S_{f-avg}^{1/2} \tag{4.23}$$

Substituting Equation (4.6) and rearranging yields the familiar form of Manning's equation:

$$Q = \frac{1.486}{n} A R^{2/3} S_{f-avg}^{1/2} \tag{4.24}$$

where:

V	=	velocity (fps)
Q	=	flow rate (cfs)
n	=	Manning's roughness coefficient (n-value) is a measure of the frictional resistance exerted by a channel on the flow.
A	=	cross sectional area of water (sq ft)
R	=	hydraulic radius = A/P
P	=	wetted perimeter (ft)
$S_{f\text{-}avg}$	=	friction slope (ft/ft)

Other helpful versions of this equation are:

$$S_{f-avg} = \frac{n^2 Q^2}{2.208 A^2 R^{4/3}} \tag{4.25}$$

$$H_f = \frac{n^2 L Q^2}{2.208 A^2 R^{4/3}} \tag{4.26}$$

Variations of these formulas can be developed for various prismatic channel shapes using the formulas provided in Section 4.2.7.

Important Note about the Friction Slope:

- **$S_{f\text{-}avg}$ ≠ So except for in uniform flow:** A common misconception in the application of Manning's equation is to assume that the bed slope of the channel, S_o, can be utilized interchangeably for the slope of the energy gradient, $S_{f\text{-}avg}$. This generalization is not correct. This substitution is correct only for the limited condition of uniform (or 'normal') flow, when the two gradients are parallel. Uniform flow rarely occurs in nature and occurs only in 'hydraulically long' engineered channels which have a constant Q, cross section, slope, and material for an extended distance. Assuming normal depth may be acceptable for preliminary sizing of storm drains, where accepted by the engineering authority, however, in most cases, $S_f \neq S_o$, and the actual depth of flow must be determined using the procedures in Section 4.4.
- **$S_{f\text{-}avg}$:** Whenever gradually varied flow conditions exist, and **$S_{f\text{-}avg} \neq S_o$**, the friction slope will be changing along the length of the system. Because of this, the analysis will use the average friction slope between two points. It is important that these two points not be too far apart to minimize errors in determining the average friction slope. See further discussion in Section 4.4.

Roughness Coefficient (n-Value)

The choice of a suitable channel roughness value (a.k.a. Manning's n coefficient or Manning's n value) for a given cross section is important for the hydraulic capacity analysis of the system. Manning's n-values vary considerably depending on the surface materials and roughness, density and type of vegetation, channel irregularity, alignment (degree of meandering), effects of silting and scouring, obstructions, channel size and shape, and depth of flow.

The Engineer must therefore make assumptions based on industry knowledge and practice, as well as professional judgement, that best represent or in many cases bracket the critical conditions that will properly inform the design of a hydraulic system or structure. **In many cases, the Engineer will need to analyze multiple n-values** representing a variety of possible scenarios that may exist across the lifecycle of the facility. For example, when a facility is new and fully maintained versus later in its life and fully vegetated; also, different n-values shall be used when considering maximum depth of flow, versus peak velocity.

4.2.6.2 Normal Depth Calculation

As discussed previously in this manual, uniform flow or 'normal depth' is a unique case where the friction slope is equal to the channel bed slope, or $S_f = S_O$. This unique case will only occur if a system is unchanged for long distances, which is relatively uncommon in practice. Determining the 'normal' depth is still a helpful reference point – or for preliminary/ rough sizing purposes, as long as it is understood that it is not necessarily the actual depth that will exist within the storm drain or channel. The 'normal' depth (uniform flow) for a channel or storm drain can be determined with the following equation:

$$Q = \frac{1.486}{n} A R^{2/3} S_O^{1/2} \qquad (4.27)$$

Where all terms are as defined previously for Manning's equation, except:

S_O = the channel longitudinal bed slope (ft/ft)

King's Method can also be used as a simplified way to calculate y_n for rectangular and trapezoidal channels, use Equation (4.28).

$$Q = \frac{K\prime}{n} b^{8/3} S_O^{1/2} \qquad (4.28)$$

which can be rearranged to:

$$K' = \frac{nQ}{b^{8/3} S^{1/2}} \qquad (4.29)$$

where:

k' = Factor obtained from Kings manual Table 7-11, which is a function of (y_n/b,z)
b = bottom width of channel (ft)
S_0 = slope of invert (ft/ft)
n = Manning's roughness coefficient
z = side slope of channel

K' can be looked up or interpolated on Table 7-11 in King's Manual and an appropriate yn/b determined (referred to as D/b in the handbook), from which the normal depth can be calculated.

The calculations for a circular channel without access to a computer program are time consuming. King's Method may be used for a non-computer solution. See Table 4.5 for mathematical equations to calculate properties of a circular channel. When solving for flowrate, depth of water or diameter of pipe, K and K' can be looked up or interpolated on Tables 7-13 and 7-14, respectively, in King's Manual for circular channels.

Variations of Manning's formula can be developed for different channel geometries and purposes. One common form for determining the 'just full' diameter D of a circular pipe is:

$$D_r = \left(\frac{2.16nQ}{\sqrt{S_o}}\right)^{3/8} \qquad (4.30)$$

4.2.6.3 N-Values for Unlined/Vegetated Engineered Channels

The following table contains guidance for use in Engineered prismatic channels that are either unlined (earthen) or vegetated.

Table 4.1: Manning's Roughness Coefficient for Unlined/Vegetated Engineered Channels

Type of Channel and Description	n-value		
	Minimum	Normal	Maximum
a. Earthen, Straight and Uniform	0.020	0.025	0.035
b. Vegetated, mowed grass	0.022	0.027	0.033
c. Vegetated, tall grass, dense weeds[1]	0.050	0.080	0.120
d. Dense brush[1]	0.080	0.100	0.140

4.2.6.4 N-Values for Engineered Channels

The following table identifies n-values for the types of Engineered channel linings that are allowed. See Table 6.1 for closed conduit n-values.

[1] Vegetated engineered channels shall use a Manning's roughness coefficient of 0.02 to compute the potential scour before establishment of vegetation for the post construction or "new channel" condition.

Table 4.2: Manning's Roughness Coefficient for Engineered Channels

Type of Channel and Description	n-value[1]		
	Minimum	Normal	Maximum
A. LINED OR BUILT-UP CHANNELS			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Unfinished	0.014	0.017	0.020
4. Shotcrete, good section	0.016	0.019	0.023
5. Shotcrete, wavy section	0.018	0.022	0.025
b. Soil Cement	0.018	0.020	0.025
c. Grouted Rock Riprap	See Section 4.2.6.5		
d. Loose Rock Riprap			
e. Gabions	0.030	0.040	0.050

1.Excerpt from: Simons, Li, and Associates (1981). Adapted from Chow (1959), and USDOT (2005).

4.2.6.5 N-Values for Rock Lined Channels

The Manning's roughness coefficient (n) for hydraulic computations shall be estimated for loose rock riprap using the Manning-Strickler equation (Equation (4.31)). Equation (4.31) (Chang, 1992) does not apply to grouted rock riprap or to very shallow flow.

$$n = 0.0395 d_{50}^{1/6} \tag{4.31}$$

where:

n = Manning roughness coefficient (dimensionless)
d_{50} = median stone diameter (feet)

For the commonly used Caltrans classes of loose rock riprap, Table 4.3 can be used, which is based on Equation (4.31).

Table 4.3: Manning's Value by Rock Class

Nominal RSP class by median particle diameter [a]		Nominal median particle weight [b] W_{50}	Median Stone Diameter [c] d_{50}		Manning n
Class	Diameter (inches)		Inches	Feet	ungrouted[d]
V	18	1/4 ton	18.8	1.6	0.043
VI	21	3/8 ton	22.0	1.8	0.044
VII	24	1/2 ton	25.3	2.1	0.045
VIII	30	1 ton	31.5	2.6	0.046
IX	36	2 ton	37.8	3.1	0.048
X	42	3 ton	44.3	3.7	0.049
XI	46	4 ton	48.4	4.0	0.050

(a) Class I -VIII is based upon Caltrans Method B Placement, which allows dumping of the rock and spreading by mechanical equipment. Local surface irregularities shall not vary from the planned grade by more than 1 foot, measured perpendicular to the slope. Class VI-XI may require special placement, see Caltrans (2016) or Greenbook for more information. (b) per Caltrans (2018). (c) Assumes specific weight of 165 lb/ft3. The designer shall take care to apply a unit weight that is applicable to the type of riprap specified for the project and adjust their calculations when necessary. (d) Based on Manning-Strickler relationship (Chang, 1988).

Where grouted riprap is used, a 20% roughness coefficient reduction (n grouted = 0.80 n ungrouted) shall be utilized for velocity-based design, such as for energy dissipation/scour minimization measures. For grouted riprap channel capacity design, the roughness coefficients in Table 4.3 shall be utilized.

4.2.6.6 N-Values for Natural, Unlined, and Composite Systems

The cross section of a natural or artificial watercourse may be composed of several distinct subsections, with each subsection having different hydraulic characteristics, such as hydraulic roughness and average flow depth. For example, a natural alluvial watercourse may have a primary, sand-bed channel bounded on both sides by densely vegetated, overbank floodplains. Or a street conveying runoff to the right of way may be bounded on both sides by landscaped front yards having shallower flow depths and slower flow velocities than that found in the paved street section.

When applying the Manning's formula to natural or composite systems, like these described above, it is necessary to compute a composite n value that accounts for irregular or inconsistent conditions that exist across the entire cross section.

Open Channel Hydraulics Section 6-5 (Chow, 1959) provides an example of computing composite roughness coefficients (n) across a single channel section as summarized below for natural systems.

In some cases, a single composite n-value may be used across the entire cross section of flow. Some hydraulic models such as HEC-RAS allow different Manning's n-values to be entered for various zones across the cross section instead of calculating one composite value. In such a case for a natural floodplain, it may still be appropriate to calculate a composite n-value for the overbanks separate from a composite n-value for the main channel.

Natural Watercourses and Unlined Channel Systems

To address all the factors affecting roughness in natural systems, composite roughness coefficients may be determined using Cowen's method, see Equation (4.32) or by Equation (4.33).

$$n = (n_b + n_1 + n_2 + n_3 + n_4)\, m \tag{4.32}$$

where:

- n = Manning's roughness coefficient including boundary and vegetation effects (dimensionless)
- n_b = base n value for the surface material of the channel or floodplain
- n_1 = value of n for the effect of surface irregularity
- n_2 = value of n for variations in shape and size of channel or floodplain
- n_3 = value of n for obstructions
- n_4 = value of n for vegetation
- m = correction factor for meandering of channel or floodplain

The parameters used for the selection of the variables in Cowen's method are summarized in Table 4.4 and were obtained from Open Channel Hydraulics (Chow, 1959) and USGS Water Supply Paper 2339.

Table 4.4: Variables for Cowen's Method of Determining n Value

Channel Conditions		Values	
Base n value	Concrete	n_b	0.012 - 0.018
	Fine Sand		0.020 - 0.028
	Firm Soil/Earth		0.022 - 0.028
	Coarse Sand		0.026 - 0.035
	Gravel		0.028 - 0.035
	Cobble		0.030 - 0.050
	Boulder		0.040 - 0.070
Degree of Irregularity	Smooth	n_1	0.00
	Minor		0.001 - 0.005
	Moderate		0.006 - 0.010
	Severe		0.011 - 0.020
Variation in channel cross section	Gradual	n_2	0.00
	Alternating occasionally		0.001 - 0.005
	Alternating frequently		0.010 - 0.015
Obstructions	Negligible	n_3	0.000 - 0.004
	Minor		0.005 - 0.015
	Appreciable		0.020 - 0.030
	Severe		0.040 - 0.050

Channel Conditions		Values	
Vegetation	Low Grasses	n_4	0.002 - 0.010
	Medium Density		0.010 - 0.025
	High Density		0.025 - 0.050
	Very Dense		0.050 - 0.100
	Willows or Similar		0.100 - 0.150
Ratio of Meandering	Minor	m	1.00
	Appreciable		1.15
	Severe		1.30

As an alternative to Cowen's method described above, in a non-urban setting, the composite roughness for a channel cross section may also be determined using Equation (4.33) below per HEC-RAS 5.0 Reference Manual and FlowMaster – Open and Closed Channel Weighting Methods.

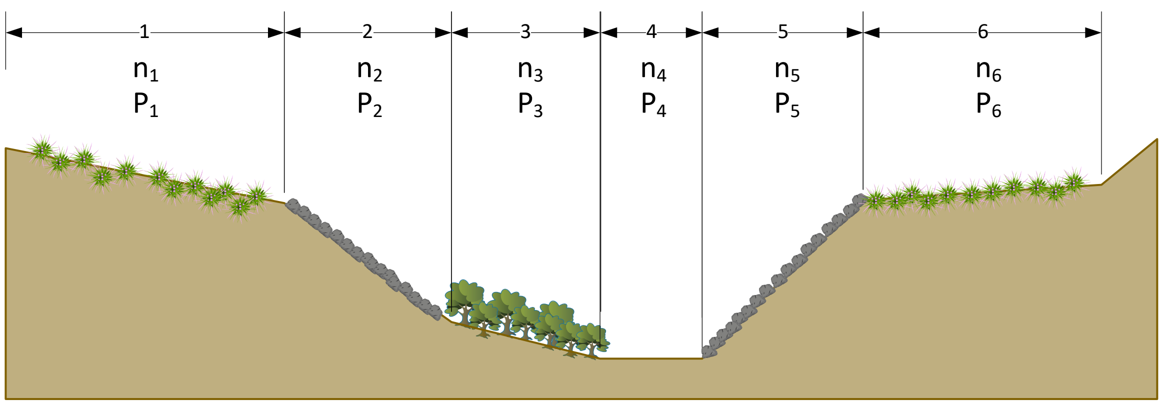
$$n = \left(\frac{\sum_{i=1}^{N}\left(P_i n_i{}^{1.5}\right)}{P}\right)^{2/3} \tag{4.33}$$

where,

n = roughness coefficient, for segment i-th segment (dimensionless)
P_i = wetted perimeter, for the i-th segment
P = overall wetted perimeter

Figure 4.5 below shows an example channel cross section that contains multiple regions with different roughness values due to different vegetation densities and type of channel lining. When using Equation (4.33), the resulting composite roughness should be compared with other methods to check for reasonableness.

Figure 4.5. Cross Section of Non-Urban Natural or Artificial Channel



Urban Floodplain Systems

In large storms in an urban setting, it is not unusual for buildings and other structures to occupy a portion of any given hydraulic cross section. Under these circumstances, it is often difficult to estimate both the effective width of the cross section and the Manning's roughness coefficient for the overbank areas.

Given this situation, the engineer should eliminate the portion of the cross section occupied by the building.

Where only an estimate of the computed water surface elevation is needed, a second option may be selected. An adjusted urban roughness coefficient, n_u, may be computed and applied to the total cross-sectional area (Hejl, 1977). See Figure 4.6 for a graphical representation of each of the variables.

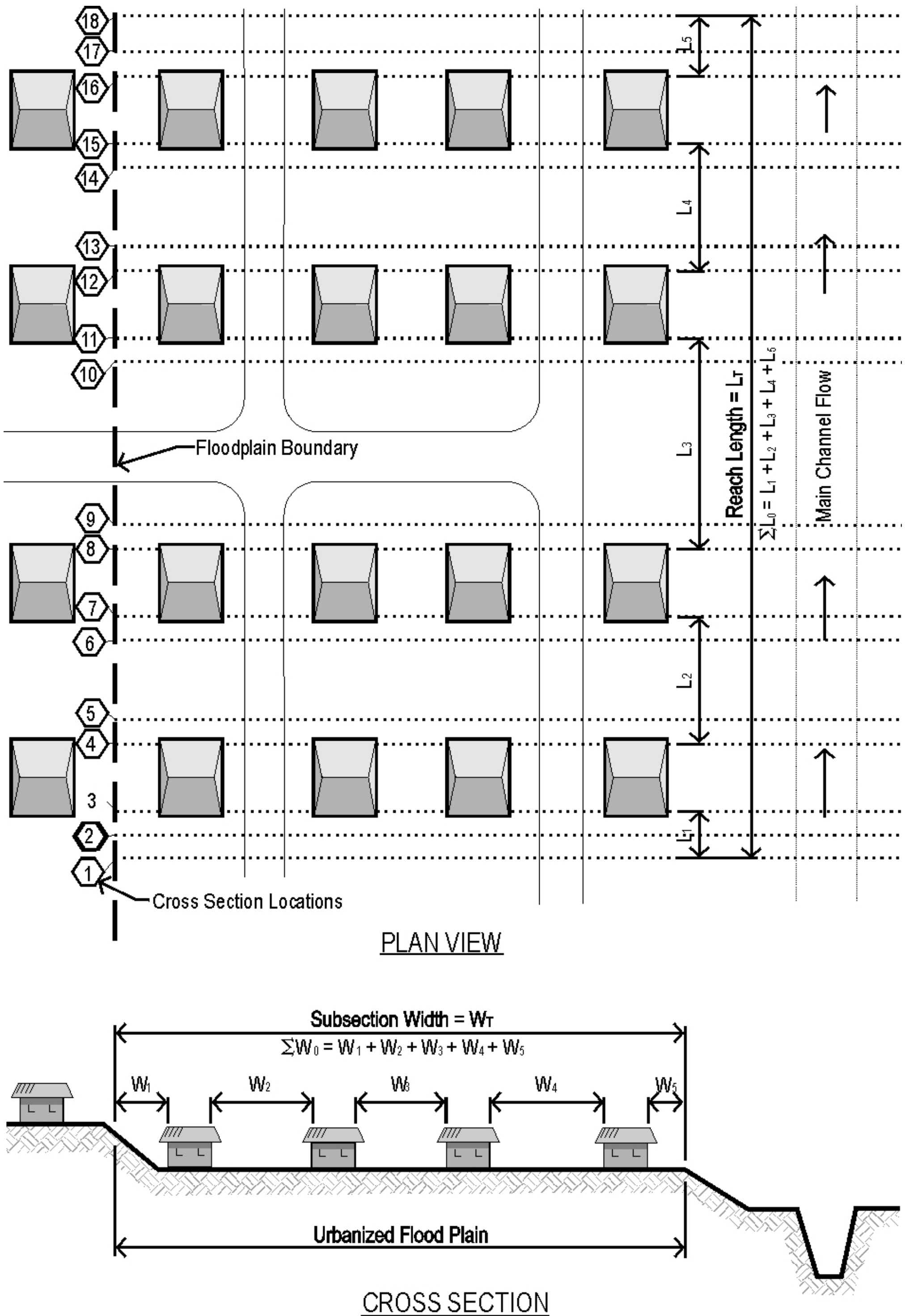
$$n_u = n_0 \left(1.5\left(\frac{W_T}{\sum W_0}\right) + \left(1 - \frac{W_T}{\sum W_0}\right)\frac{\sum L_0}{L_T} - 0.5\right) \tag{4.34}$$

where:

n_u	=	adjusted urban roughness coefficient (dimensionless)
n_0	=	roughness coefficient for the area between the buildings in the floodplain (dimensionless)
W_T	=	total width of the floodplain including buildings (ft)
W_0	=	sum of clear width between buildings, measured perpendicular to flow (ft)
L_0	=	sum of individual length between buildings measured parallel to flow (ft)
L_T	=	total length of the floodplain including buildings (ft)

Figure 4.6: Diagram of Urban Floodplain

(Hejl, 1977, Journal of Research, U.S. Geological Survey)



4.2.7 Cross Section Properties for Prismatic Channels

Table 4.5 lists the algebraic expressions for computing the area and hydraulic radius as used in Manning's equation for typical channel sections.

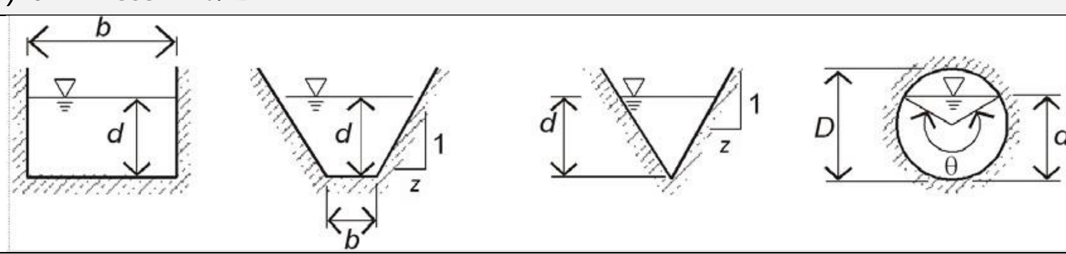
Table 4.5: Elements of Channel Sections (1)

Channel Section	Area	Wetted Perimeter	Top Width	Hydraulic Radius
Rectangle	bd	$b+2d$	b	$\frac{bd}{b+2d}$
Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$b+2zd$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$2zd$	$\frac{zd}{2\sqrt{z^2+1}}$
Circular < 1/2 full (2)	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$Dsin\theta$ or $2\sqrt{d(D-d)}$	
Circular > 1/2 full (3)	$\frac{D^2}{8}\left(2\pi\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$Dsin\theta$ or $2\sqrt{d(D-d)}$	$\left(\frac{45D}{\pi(360-\theta)}\right)*\left(2\pi-\frac{\pi\theta}{180}+sin\theta\right)$

(1) After USDA Soil Conservation Service ES-33 (NRCS), 1956.

(2) $\theta = 4\sin^{-1} d/D$ Insert θ in degrees

(3) $\theta = 4\cos^{-1} d/D$ Insert θ in degrees



Rectangle Trapezoid Triangle Circular

4.3 ENERGY LOSSES

The headlosses that need to be determined are friction, transition, junction, manhole, bend, inlet, and exit. These losses need to be determined individually and then added together to determine the overall headloss for each segment of the storm drain. The methods for determining the various headlosses presented in this Section were selected for their wide acceptance and ease of use.

4.3.1 Friction Losses

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

$$h_f = S_{f-avg}L \tag{4.25}$$

where:

h_f = friction headloss (ft)
$S_{f\text{-}avg}$ = average friction slope across the reach length (ft/ft)
L = reach length (ft)

4.3.2 Junction Losses

A junction occurs where one or more lateral systems enter the main storm drain/channel at a formed junction or at a manhole. Significant headlosses can occur at a junction due to both the geometry of the structure itself and the confluencing of flow from multiple directions. Junctions are therefore a form of 'Rapidly Varied Flow'. Junctions should be designed to allow flow to confluence as smoothly as possible to avoid high headlosses. Figure 4.7 through Figure 4.9 show typical junctions in plan and profile. Note that smaller confluence angles will result in less junction headloss, but this comes with the tradeoff of requiring a larger junction structure due to the long side opening required in the main line. Larger confluence angles result in higher junction headloss but require a shorter junction structure. Generally, where the lateral flows are small, the junction losses will not be significant and larger confluence angles can be used.

Figure 4.7: Storm Drain Junction

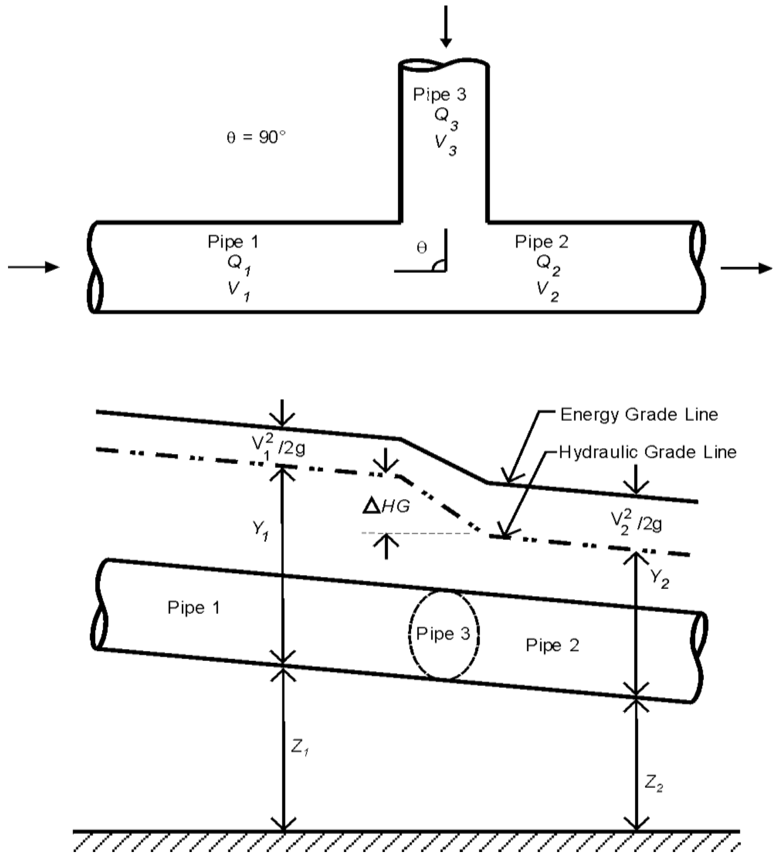


Figure 4.8: Storm Drain Junction at Manhole with Aligned Soffits Under Presure Flow

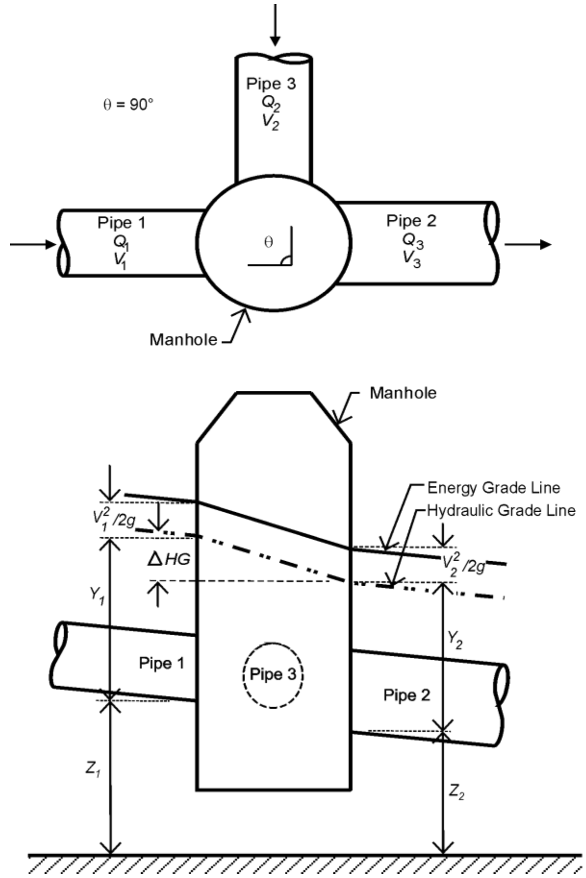
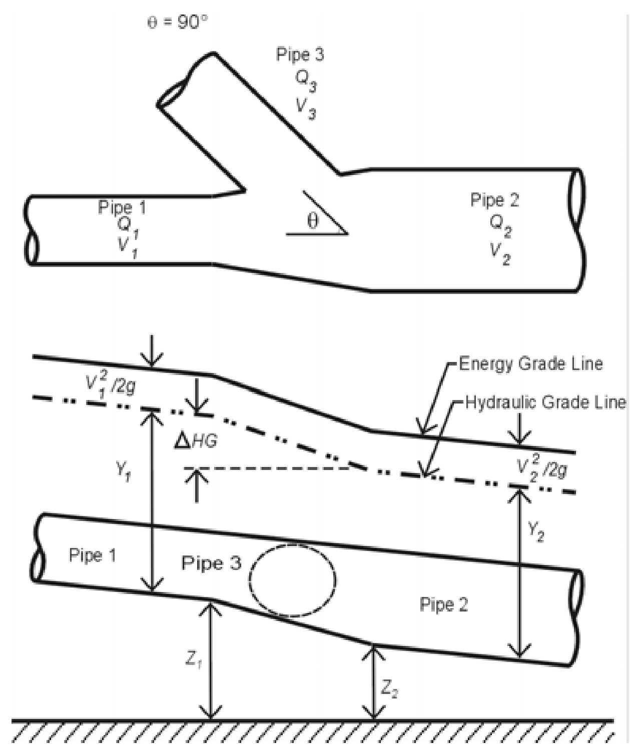


Figure 4.9: Formed Storm Drain Junction with Aligned Soffits Under Presure Flow



The Thompson Equation (Equation (4.35)), a form of the momentum equation, is used to determine the change in hydraulic grade ΔHG across a junction.

$$\Delta HG \frac{A_1 + A_2}{2} = \frac{Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta}{g} \tag{4.35}$$

or

$$\Delta HG = \frac{\left(\frac{Q_2V_2 - Q_1V_1 - Q_3V_3Cos\,\theta}{g}\right)}{\left(\frac{A_1 + A_2}{2}\right)}$$

where:

ΔHG	=	difference in upstream and downstream hydraulic grade line elevations (ft)
A_1	=	upstream flow area (sq ft)
A_2	=	downstream flow area (sq ft)
Q_1	=	upstream flowrate (cfs)
Q_2	=	downstream flowrate (cfs)
Q_3	=	lateral flowrate (cfs)
V_1	=	upstream flow velocity (ft/sec)
V_2	=	downstream flow velocity (ft/sec)
V_3	=	lateral flow velocity (ft/sec)
θ	=	angle between lateral and main line storm drain (See Figure 4.9), (degrees)

To determine junction headloss h_j, the change in velocity head must be added to the change in hydraulic grade. Assuming transition and friction losses at the junction are negligible, Equation (4.36) should be used to determine the junction headloss h_j.

$$\frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\,\theta°)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g = h_j \tag{4.36}$$

Should transition and/or friction losses be determined not to be negligible the total loss across the junction structure h_T can be determined by the following:

$$h_T = h_j + h_f + h_{tr} \tag{4.37}$$

where:

h_T	=	total loss across the junction structure (ft)
h_j	=	junction headloss per Equation (4.36) (ft)
h_f	=	friction headloss (see Section 4.3.1)
h_{tr}	=	transition headloss (see Section 4.3.3)

4.3.3 Transition Losses

Transition structures, where storm drain/channel dimensions expand or contract, introduce headloss to the system. Figure 4.10 shows the typical type of transition that can be encountered. Most references equate losses in transitions directly to change in velocity head through use of energy coefficients as described below.

The headloss due to the **expansion** of flow (velocity decreases) is expressed as:

$$h_t = k_o \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \tag{4.38}$$

where:

h_t = transition headloss (ft)
k_o = coefficient for transition loss due to expansion
V_1 = upstream velocity (ft/sec)
V_2 = downstream velocity (ft/sec)
g = acceleration due to gravity 32.2 ft/sec ^{2}
Note: V_1 is greater than V_2

The values for the transition coefficient, k_o, for enlargements are given in Table 4.6.

The headloss due to the **contraction** of flow (velocity increases) is expressed as:

$$h_t = k_i \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \tag{4.39}$$

where:

k_i = coefficient for transition loss due to contraction
V_1 = upstream velocity (ft/sec)
V_2 = downstream velocity (ft/sec)
g = acceleration due to gravity 32.2 ft/sec ^{2}
Note: V_2 is greater than V_1

Values for the transition loss coefficient, k_i, for contractions can also be found in Table 4.6.

Figure 4.10: Transition Loss

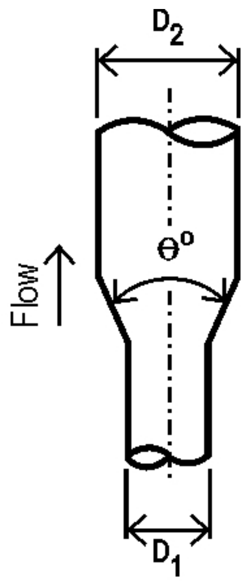


Table 4.6: Storm Drain Energy Loss Coefficient Under Open Channel Conditions

Shape	K_o	K_i
Straight Line* 10°	0.20	0.10
Straight Line* 15°	0.30	0.10
Straight Line* 20°	0.40	0.10
Straight Line* 30°	0.70	0.10
Warped Design**	0.20	0.10

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

4.3.4 Minor Losses

4.3.4.1 Straight-Through Manhole Losses (no laterals)

In a straight-through manhole where there is no change in pipe size or rate of flow, the loss can be estimated from Equation (4.40) below such as for District Manhole No. 1 (District Standard Drawing MH251). For manholes with the same shape as the conduit, there are no manhole losses. Where a change in pipe size and/or change in flowrate occurs, no additional head loss need be calculated for the manhole. It is considered to be included in the transition and/or junction loss.

$$h_{mh} = 0.05\frac{V^2}{2g} \tag{4.40}$$

where:

h_{mh} = headloss due to a manhole (ft)
V = velocity (ft/sec)

4.3.4.2 Bend Losses (no laterals)

The bend loss of a curved storm drain is determined using Equation (4.41). Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

$$h_b = k_b\frac{V^2}{2g} \tag{4.41}$$

where:

h_b = headloss due to a bend (ft)
k_b = bend loss coefficient = $0.20\sqrt{\frac{\Delta}{90}}$
V = velocity (ft/sec)
g = acceleration due to gravity 32.2 ft/sec ^{2}
Δ = central angle of bend, not to exceed 90 degrees (See Figure 6.1)
θ = central angle of bend, not to exceed 90 degrees (See Figure 6.1)

4.3.4.3 Angle Point Losses

Angle point losses are calculated using Equation (4.42). This formula should be used where the mainline horizontal alignment has an angle point, such as at a manhole or collar, but there are no laterals.

$$h_{a\,pt.} = k_b\frac{V^2}{2g} \tag{4.42}$$

where:

$h_{a.\,pt.}$ = headloss due to a bend (ft)
$k_{a.\,pt.}$ = bend headloss coefficient
V = velocity of flow in the bend (ft/sec)

The value of the bend loss coefficient, k_b, depends upon the angle of the bend. It can be estimated from Equation (4.43)

$$k_{a.\ pt.} = 0.0033\theta \tag{4.43}$$

where:

$k_{a.\,pt.}$ = angle point headloss coefficient
θ = angle of curvature or deflection (degrees) not to exceed 6 degrees without prior approval from the District (See Figure 6.1)

4.3.4.4 Inlet Losses

At open inlets to a storm drain system, for example where a natural watercourse or roadside ditch is intercepted, or a basin drains into an underground storm drain, an inlet will function the same as a culvert inlet. Under inlet control, the hydraulic grade line at the entrance can be estimated by using the appropriate procedures and figures presented in the Culvert Chapter. Under outlet control, inlet losses can be calculated using Equation (4.44).

$$h_i = k_{en}\frac{V^2}{2g} \tag{4.44}$$

where:

h_i = headloss at inlet (ft)
k_{en} = entrance loss coefficient

The k_{en} in the equation is equivalent to k_e values listed in Table 7.1.

In addition to the entrance loss, losses associated with a protection barrier or trash rack over the inlet should be taken into consideration. Procedures to estimate headlosses due to barriers or trash racks can be found in Section 9.7.

4.3.4.5 Exit Losses

When a storm drain outfalls to a retention basin, lake, or natural or engineered open channel, additional headloss occur at the outlet in the form of expansion losses due to the change in velocity at the outlet of the pipe, and due to the changes in flow direction. The exit headloss at storm drain outlets is expressed as (Clark County Regional Flood Control District, 1990):

$$h_o = 1.0\frac{V_O^2}{2g} \tag{4.45}$$

where:

h_O = headloss at outlet (ft)
V_O = average outlet velocity (ft/sec)

4.3.5 <u>Hydraulic Jumps</u>

A hydraulic jump occurs when flow changes rapidly from supercritical flow to subcritical flow, such as: 1) when the slope of a channel changes from steep to mild; 2) at sudden expansions or contractions in the channel section; 3) at locations where a barrier, such as a culvert or bridge, occurs in a channel of steep slope; 4) at the downstream side of dip crossings or culverts; and 5) where a channel of steep slope discharges into other channels.

Hydraulic jumps dissipate energy, and consequently drainage outlet structures and drop structures are often designed to force a hydraulic jump as an efficient way to minimize the erosive potential of floodwaters. However, because of the high turbulence associated with hydraulic jumps, they must be contained within a well-protected area. Complete computations must be made to determine the height, length, and other characteristics of the jump (including consideration of a range of flows) in order to adequately size the containment area.

The type of hydraulic jump that forms and the amount of energy that it dissipates is dependent upon the upstream (drop face) Froude number (F_{r1}). The various types of hydraulic jumps that can occur are listed in Table 4.7.

Table 4.7: Types of Hydraulic Jumps
(USDOT, FHWA, HEC-14, 2006)

Upstream Froude Number	Type of Jump	Energy Loss, %
1.0 < $F_{r1} \leq$ 1.7	Undular Jump – not effective energy dissipation. Undular jumps are to be avoided.	Minimal
1.7 < $F_{r1} \leq$ 2.5	Weak Jump – not effective energy dissipation	20%
2.5 < $F_{r1} \leq$ 4.5	Oscillating Jump – can cause erosion problems downstream of jump	20 to 45
4.5 < $F_{r1} \leq$ 9.0	Steady Jump – fluid turbulence is mostly confined to jump, and downstream water surface is comparatively smooth	45 to 70
9.0 < F_{r1}	Strong Jump – highly efficient jump results but the rough water surface may cause downstream erosion problems	70 to 85

4.3.5.1 Jump Height / Conjugate Depth

The subcritical depth of flow immediately downstream of a hydraulic jump is referred to as the sequent or conjugate depth (Y_2). The initial (upstream) supercritical flow depth (Y_1) and sequent depth (Y_2) have the same Specific Energy. The conjugate depth in rectangular channels whose upstream Froude number is > 1.7, can be computed by use of the following equation:

$$Y_2 = \frac{1}{2} Y_1 \left[\sqrt{1 + 8F_{r1}^2} - 1 \right] \quad (4.46)$$

For hydraulic jumps at a drop structure, calculate the required tailwater depth (Y_2) using Equation (4.46). Compare the results of this calculation to the modeled tailwater depth determined in the subcritical HEC-RAS model. The modeled tailwater depth must be greater than or equal to the calculated required headwater depth for a hydraulic jump to start near the toe of the drop. If the modeled tailwater depth is less than required, the drop structure geometry must be re-evaluated.

The solution for conjugate depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation (4.47), which is derived from momentum equations. It is also acceptable for design purposes to determine the sequent depth in trapezoidal channels from Equation (4.46). Equation (4.46) is much simpler to solve and produces only slightly greater values for sequent depth than does Equation (4.47).

$$\frac{ZY_1^3}{3} + \frac{ZY_1^2}{2} + \frac{Q}{gA_1} = \frac{ZY_2^3}{3} + \frac{bY_2^2}{3} + \frac{Q}{gA_2} \tag{4.47}$$

where:

Z = Channel side slope horizontal distance per foot of drop
Y_1 = initial (upstream) flow depth (may be taken from cross section at drop toe, supercritical HEC-RAS model) (ft)
Q = discharge (cfs)
A_1 = area upstream of the jump (sq ft)
Y_2 = the tailwater depth required to cause a jump to form immediately downstream of the initial depth location for Y_1 (also called the sequent depth) (ft)
b = bottom width (ft)
A_2 = area downstream of the jump (sq ft)

4.3.5.2 Jump Length

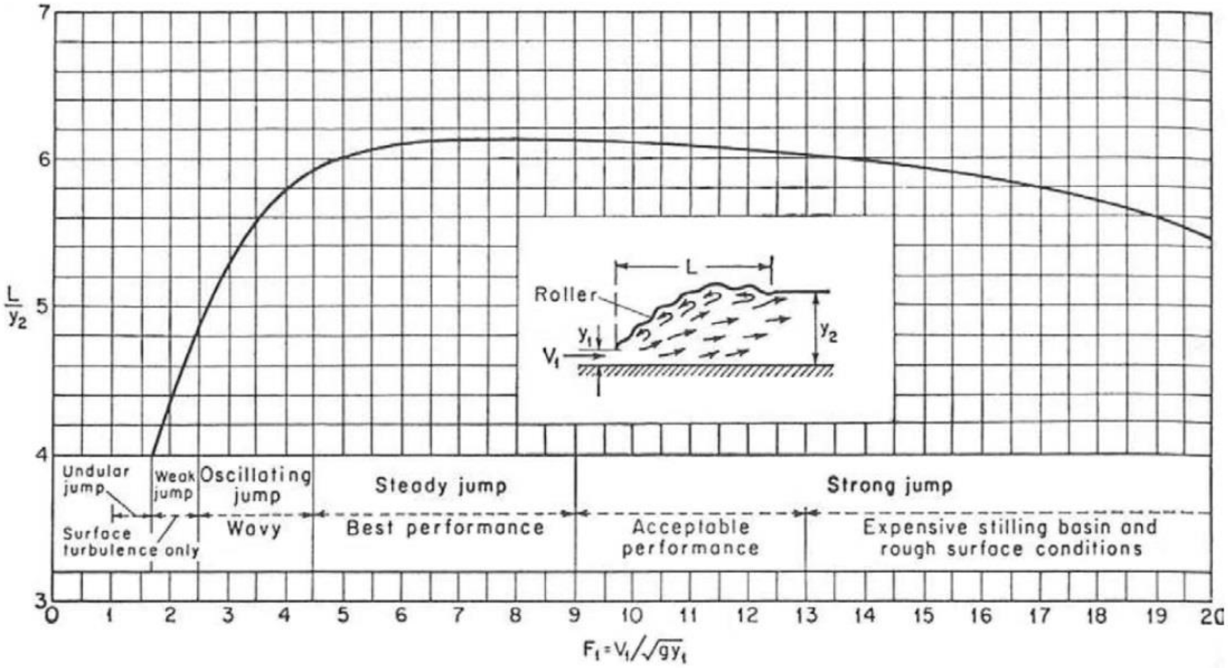
The length of a hydraulic jump is defined as the distance from the front face of the jump to a point immediately downstream of the roller. After the hydraulic jump has been analyzed using the guidelines provided in Section 4.3.5.1, the jump length must be calculated. For certain hydraulic structures, this will aid the designer in determining the appropriate stilling basin length and the need for additional rock lining downstream of the end sill. The following values are required to determine the hydraulic jump length:

Y_2 = the tailwater depth required to cause a jump to form immediately downstream of the initial depth location for Y_1 (also called the sequent depth) (ft)
F_1 = Froude Number = $V_1/(gy_1)^{1/2}$ based on depth and velocity at drop toe

Use the above values to determine the length of the hydraulic jump in Figure 4.11. Note that this figure is for horizontal channels, which is appropriate for most applications. Curves for sloping channels (from 5 to 25%) are in Chow, 1959.

Figure 4.11: Length in Terms of Sequent Depth of Jumps in Horizontal Channels

(Source: U.S. Bureau of Reclamation, 1955)



4.3.5.3 Jump Location

Hydraulic Jumps will occur at a location where the P+M of the supercritical profile is equal to P+M of the subcritical profile.

$$\left[\left(A_1 \cdot y_{cg_1}\right) + \frac{Q^2}{g \cdot A_1}\right]_{\boldsymbol{super}critical} = \left[\left(A_2 \cdot y_{cg_2}\right) + \frac{Q^2}{g \cdot A_2}\right]_{\boldsymbol{sub}critical} \quad (4.48)$$

(P + M)super > (P + M)sub → Jump occurs on Subcritical slope
(P + M)super < (P + M)sub → Jump occurs on Supercritical slope
(P + M)super = (P + M)sub → Jump occurs at grade break

We can simplify these above equations and derive that the hydraulic jump will occur at a location where the supercritical conjugate depth is equal to the subcritical depth.

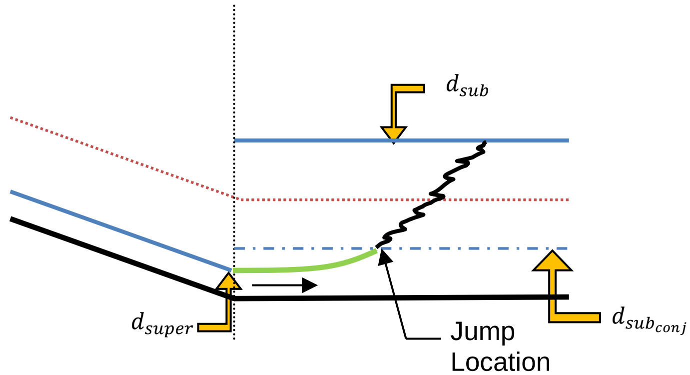
$$d_{super_{conj}} = d_{sub}$$

When this condition is met, the converse will also be true; i.e. the subcritical conjugate depth will be equal to the supercritical depth.

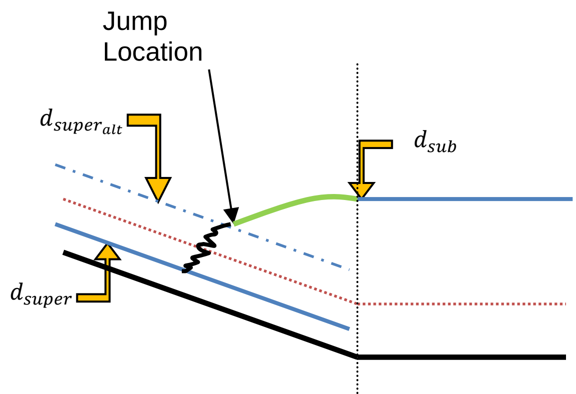
$$d_{sub_{conj}} = d_{super}$$

If these conditions are met at the grade break, the jump will occur there. Otherwise, the jump will occur upstream or downstream of the grade break at a location where this condition is eventually met. To determine which slope the jump will occur on and where, start by calculating the conjugate depth for the subcritical profile, $d_{sub_{conj}}$

- If $d_{sub_{conj}}. > d_{super}$ the jump will occur on the downstream slope. A supercritical gradually varied flow curve will extend downstream from d_{super} at the grade break and will continue until $d_{sub_{conj}}$ is reached. The jump will occur at this location.



- If $d_{sub_{conj}}. < d_{super}$ the jump will occur on the upstream slope. Start by calculating $d_{super_{conj}}$. A subcritical gradually varied flow curve will extend upstream from d_{sub} at the grade break, and will continue until $d_{super_{conj}}$ is reached. The jump will occur at this location.



An Example problem for hydraulic jump calculations can be found in Section 4.4.5.1.

4.3.6 Piers

The effect of piers on open channel design shall be considered at bridge crossings (that have piers) and where an open channel or box conduit not flowing full discharges into a length of multi-barreled box. This effect is especially important when flow is supercritical, and when transported debris impinges on the piers.

The total pier width shall include an added width for design purposes to account for debris. Inasmuch as the debris width to be used in design will vary with each particular situation, the District shall be contacted during the preliminary design stages of a project for a determination of appropriate width. Streamline piers should be used when heavy debris flow is anticipated.

The water surface elevations at the upstream end of the piers shall be determined by equating pressure plus momentum. The water surface profile within the pier reach shall be determined by the Bernoulli equation. The water surface elevations at the downstream end of the piers may be determined by applying either the pressure plus momentum equation or the Bernoulli equation.

Pressure + Momentum Equation as Applied to Bridge Piers

Based on observations of bridge pier losses it's been found that there is a loss of momentum caused by impact against the pier which produces a loss in momentum equal to M_1 (A_p/A_1). Therefore, the pressure plus momentum ($P_1 + M_1 – P_p$) should be reduced by the loss M_1 (A_p/A_1) which changes the momentum term to M1 (A_1 -A_p/A_1) = M_1 (A_2/A_1).

$$P_1 + M_1 \frac{A_2}{A_1} - P_p = P_2 + M_2 \tag{4.49}$$

$$P_3 + M_3 = P_4 + M_4 - P_p \tag{4.50}$$

where:

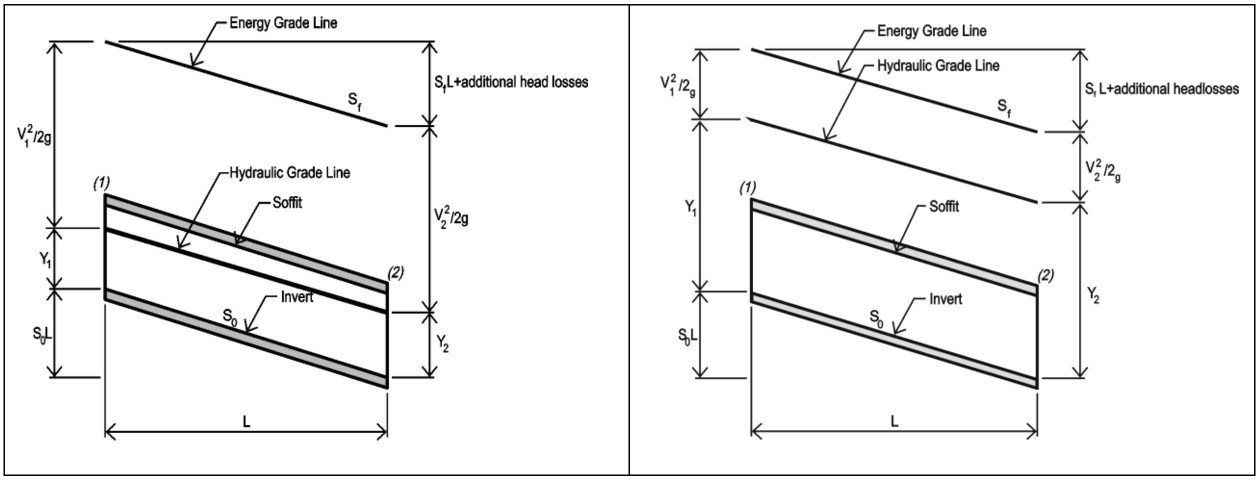
P_1 = hydrostatic pressure in unobstructed channel
M_1 = kinetic momentum in unobstructed channel
A_1 = area of unobstructed channel
A_2 = area of water within bridge = A_1 - A_p
P_2 = hydrostatic pressure within bridge based on net flow area
M_2 = kinetic momentum within bridge based on net flow area
P_P = hydrostatic pressure of bridge pier = Kp Ap Yp
A_P = area of piers
Y_P = centroid moment arm of Ap about the hydraulic grade at the section
K_P = pier factor

4.4 WATER SURFACE PROFILES

As discussed previously, uniform or 'normal' flow conditions rarely exist in storm drainage systems; in reality, the friction slope and 'losses' will be variable, and as a result, the depth and velocity will be changing along the length of the conveyance. Water surface profile calculations are performed iteratively to determine the actual depth profile (and associated velocity), and to size an acceptable engineered conveyance system.

The **depth** is important for determining whether a given flow rate will be contained within the system, or whether the system is at risk of overtopping. Based on the geometry and topography of the system/area, knowing the depth of flow can also help in determining the width of any flooding that may occur. In the case of underground storm drains, the system may flow 'open' when the depth is less than the height of the conduit, or 'pressurized', where the conduit is flowing full and a pressurized hydraulic grade line forms above the soffit.

Figure 4.12: Storm Drain Profile Flow Conditions



The **velocity** of flows is also important to understand as this will help determine whether the flood flows are likely to scour or damage the channel lining, and to assist in selecting and designing an appropriate engineered lining when needed.

Section 4.1.2 discussed the various classifications of flows that can exist. For the purposes of this manual, the following scenarios are used in studies conducted by or submitted to the District.

Table 4.8: Scenario and Associated Flow Condition

Scenario	Flow Condition	Water Surface Method
Street Flow (See Chapter 5	Steady	Uniform/Normal Flow
Underground Storm Drain (See Chapter 6	Steady or Unsteady	Varied Flow
Culverts (See Chapter 7	Steady	Varied Flow or Nomographs
Open Channels – natural or engineered (See Chapter 8	Steady or Unsteady	Varied Flow

There are several steps to determining a flow profile:

1. Divide the system into ‘reaches’.
2. Calculate the Flow Regime, including Critical and Normal Depth for each reach.
3. Determine Control Points and where GVF and RVF will occur.
4. From the Control Points, calculate GVF curves.
5. Calculate Rapidly Varied Flow (where applicable).

4.4.1 Step 1: Establish 'Reaches'

Analyze the system to identify reaches where the flow rate, slope, cross section, and material / n-value are constant. In the example below, five reaches are established:

Figure 4.13: Example Problem Establish Reaches

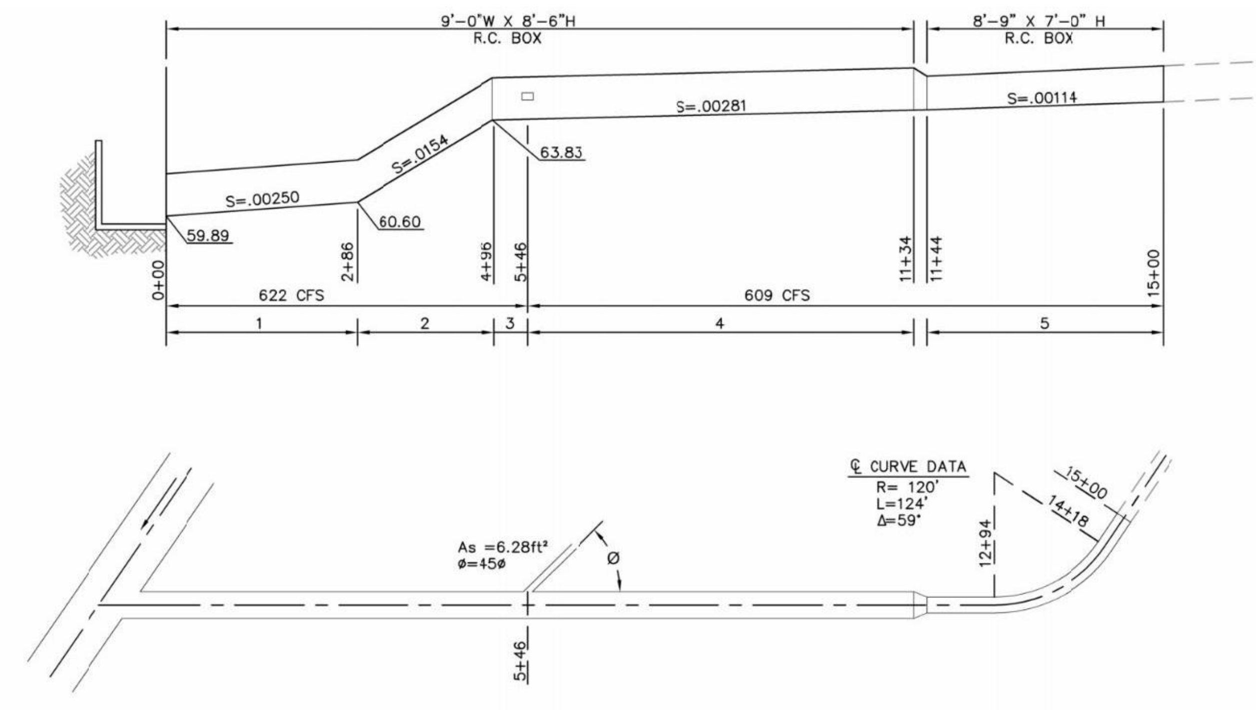


Table 4.9 : Example Problem Reaches

Reach	Station Range	Flow Rate	Cross Section	Slope	Material/ n-Value
1	0+00 – 2+86	622 cfs	RCB 9.0x8.5	0.00250	Concrete n=0.014
2	2+86 - 4+96	622 cfs	RCB 9.0x8.5	0.01540	Concrete n=0.014
3	4+96 - 5+46	622 cfs	RCB 9.0x8.5	0.00281	Concrete n=0.014
4	5+46 - 11+34	609 cfs	RCB 9.0x8.5	0.00281	Concrete n=0.014
5	11+44 - 15+00	609 cfs	RCB 8.75x7.0	0.00114	Concrete n=0.014

4.4.2 Step 2: Determine Flow Regimes

For each reach it is necessary to determine the flow regime, which requires determination of Critical Depth and Normal Depth.

- For Critical Depth, follow the methods described in Section 4.1.2.4.
- For Normal Depth, use Manning's equation as described in Section 4.2.6.2.

Wherever the normal depth is above critical depth, the water will flow subcritical, and wherever normal depth is below critical depth, the water will be flowing supercritical. Note these reaches of subcritical and supercritical flow.

For an underground storm drain, once the depth equals the conduit height the system will become pressurized. Flow in pressurized gravity flow storm drain systems is subcritical.

4.4.2.1 Example Problem

Figure 4.14: Example Problem Determine Flow Regimes

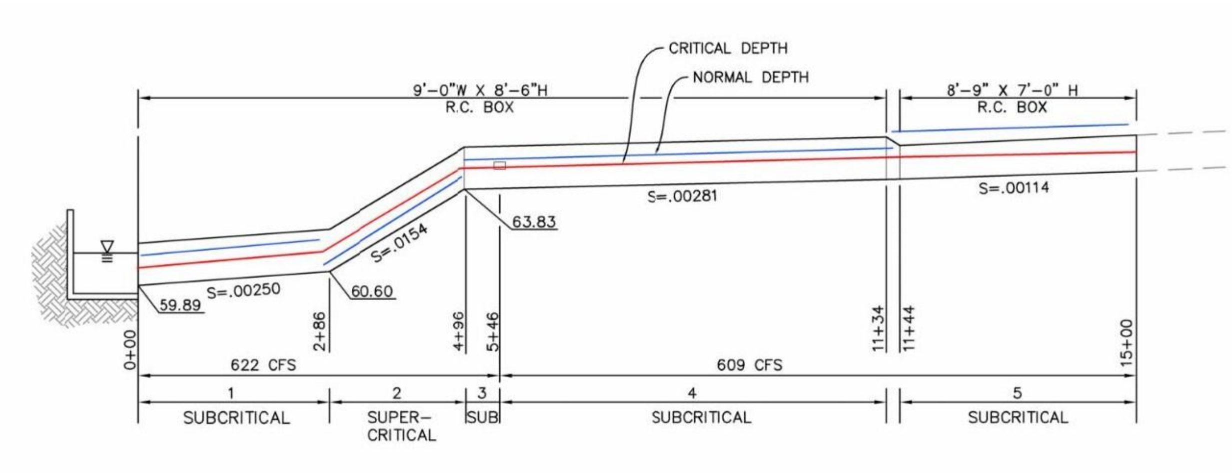


Table 4.10: Example Problem Results Summary

Reach	Section	Q (cfs)	$y_c = \left(\frac{Q^2}{b^2 g}\right)^{1/3}$	$K' = \frac{nQ}{b^{8/3} S^{1/2}}$	Yn/b Table 7-11 King's	Yn (ft)	Flow Regime
1	9'-0"W x 8'-6"H	622	5.30	0.498	0.75	6.75	Subcritical
2	9'-0"W x 8'-6"H	622	5.30	0.201	0.38	3.42	Supercritical
3	9'-0"W x 8'-6"H	622	5.30	0.469	0.71	6.39	Subcritical
4	9'-0"W x 8'-6"H	609	5.22	0.459	0.70	6.30	Subcritical
5	8'-9"W x 7'-0"H	609	5.32	0.775	1.07	9.36	Subcritical

4.4.3 Step 3: Determine Control Points and GVF/RVF Zones

Based on the calculated critical and normal depths, the control points and zones where Gradually Varied and Rapidly Varied Flow can be determined.

4.4.3.1 Control Points

A control point is any specific location where depth of flow is known, such as:

- A fixed water surface for a body of water such as a storage facility, a lake or reservoir
- Where water is known to pass through critical depth, such as:
    - A grade break where water transitions from subcritical to supercritical
    - A free outfall (water flows freely out of the system with no tailwater effects)
- Depth of flow over a weir
- Depth of flow under a gate
- A calculated depth from an upstream or downstream reach (a natural watercourse or open channel either improved or unimproved or a closed conduit)

The control points will be used as the starting point for your varied flow calculations.

The hydraulic analysis of a *supercritical* system will begin at the upstream control point and extend downstream. The upstream controlling water surface is usually referred to has the headwater.

The hydraulic analysis of a *subcritical* system will begin at the downstream outfall / control point and extend upstream. The downstream controlling water surface elevation at this location is commonly referred to as the tailwater. At the outfall, one of several conditions will be encountered: for example, a closed conduit, another open channel, a reservoir, lake or detention facility, or a free outfall such as that created by a weir. The tailwater elevation criteria described here are for determining storm drain HGL and EGL elevations only; Section 9.3 describes tailwater elevation criteria for energy dissipation calculations.

For free outfalls, the initial water surface elevation (tailwater) shall be assumed equivalent to the critical depth in the channel.

Where the channel outfalls to another drainage facility, such as a drainage channel, reservoir, or detention facility, the initial water surface elevation shall generally be set at the water surface elevation calculated for the downstream facility. The tailwater elevation shall be the water surface elevation of the channel coinciding with same return period as the channel design peak discharge.

For *conduit outfalls* into other drainage facilities, it shall be assumed that the tailwater elevation at the storm drain outlet is equivalent to the water surface elevation within the receiving channel or facility which has the same return period as the storm drain design discharge, unless otherwise approved by the governing agency. In general, the three types of tailwater conditions are:

1. Tailwater elevation above the soffit elevation. In such situations the control shall conform to the following criteria:
    a. In the case of a conduit discharging into a storage basin, the control shall be the storage basin water surface elevation coinciding with the design peak flow to the storage basin.
    b. In the case of a conduit discharging into an open channel, the tailwater elevation shall be the water surface elevation of the channel coinciding with same return

period as the storm drain design peak discharge.

c. In the case of a conduit discharging into another conduit, the control shall be the highest hydraulic grade line elevation of the outlet conduit immediately upstream or downstream of the confluence (at the upstream or downstream end of the junction structure).

2. Tailwater elevation at the crown (soffit) elevation. The tailwater shall be the soffit elevation at the point of discharge.

3. Tailwater elevation below the soffit elevation. In such situations the control shall be higher of either 1) critical depth or 2) the downstream water surface elevation.

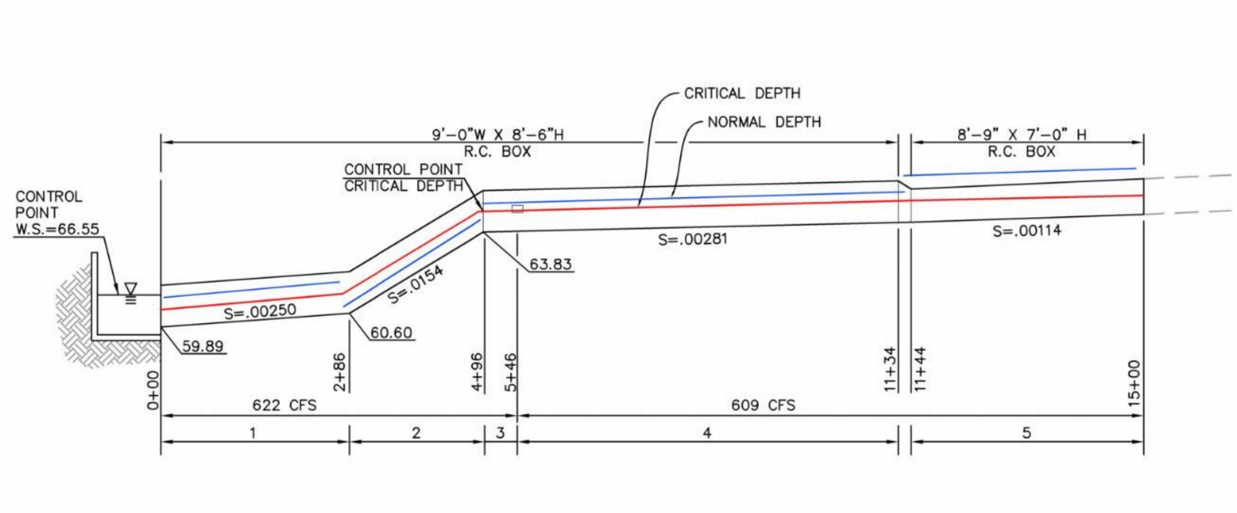
4. Where there is a mapped floodplain or floodway shown on the applicable FIRM or County floodplain map for the watercourse, the 100-year tailwater water surface elevation (WSE) shall be the floodway elevation or, if there is no floodway, the base flood elevation shown on the FIRM or County floodplain map plus 1.00 foot. The increase in control WSE depth accounts for the increase in flood elevations due to future encroachment into the floodplain.

If no accurate water surface elevation is known at the controlling end (downstream for sub-critical and upstream for super-critical flow) of the reach under consideration, an arbitrary elevation may be assumed at a cross section far enough away from the controlling end of the study reach to compensate for any initial error.

Determining how far upstream or downstream of a given location to begin the hydraulic analysis when the starting water surface elevation isn’t known is an iterative procedure. For instance, in the case of sub-critical flow within a watercourse traversing through a proposed subdivision, the modeler may choose to include additional cross sections extending 500 feet downstream of the subdivision boundary. The analysis is performed, and a water surface elevation is established at the subdivision boundary. Subsequently, the analysis is extended further downstream a reasonable distance (such as another 300-500 feet) and the results compared with the previous analysis. If the water surface elevation at the subdivision boundary remains unchanged, then the original assumption of 500 feet is adequate. If the water surface elevation changes, then this process is repeated until a consistent water surface elevation is established at the boundary.

With experience, the modeler will recognize hydraulic controls where further analysis will result in no difference for the location of interest. An example would be a road fill crossing that overtops downstream of the subdivision described above. In this case, water could pass from sub-critical to super-critical depth as it overtops the road. Where the water passes below critical depth it changes from downstream to upstream control, and thus, further analysis in the downstream direction isn’t warranted.

Figure 4.15: Example Problem Control Points



4.4.3.2 GVF and RVF

For each reach, it can be helpful to identify *conceptually* where the gradually varied flow curves will be, and where rapidly varied flow will exist.

Flow in each reach will always begin at a control point and transition toward normal depth. Normal depth will only be actually reached if the reach is unchanged for a long enough distance. Any time the geometry, slope, or flow rate changes, the targeted normal depth will change, and the water will begin transitioning toward that new normal depth. The 'transition' will occur either via Gradually Varied Flow or Rapidly Varied Flow.

Rapidly varied flow exists in locations that are highly turbulent, such as

- where water transitions from supercritical to subcritical (via a hydraulic jump), or
- where large external forces act on the flow, such as at a junction.

Gradually varied flow is the most common condition in engineered channels.

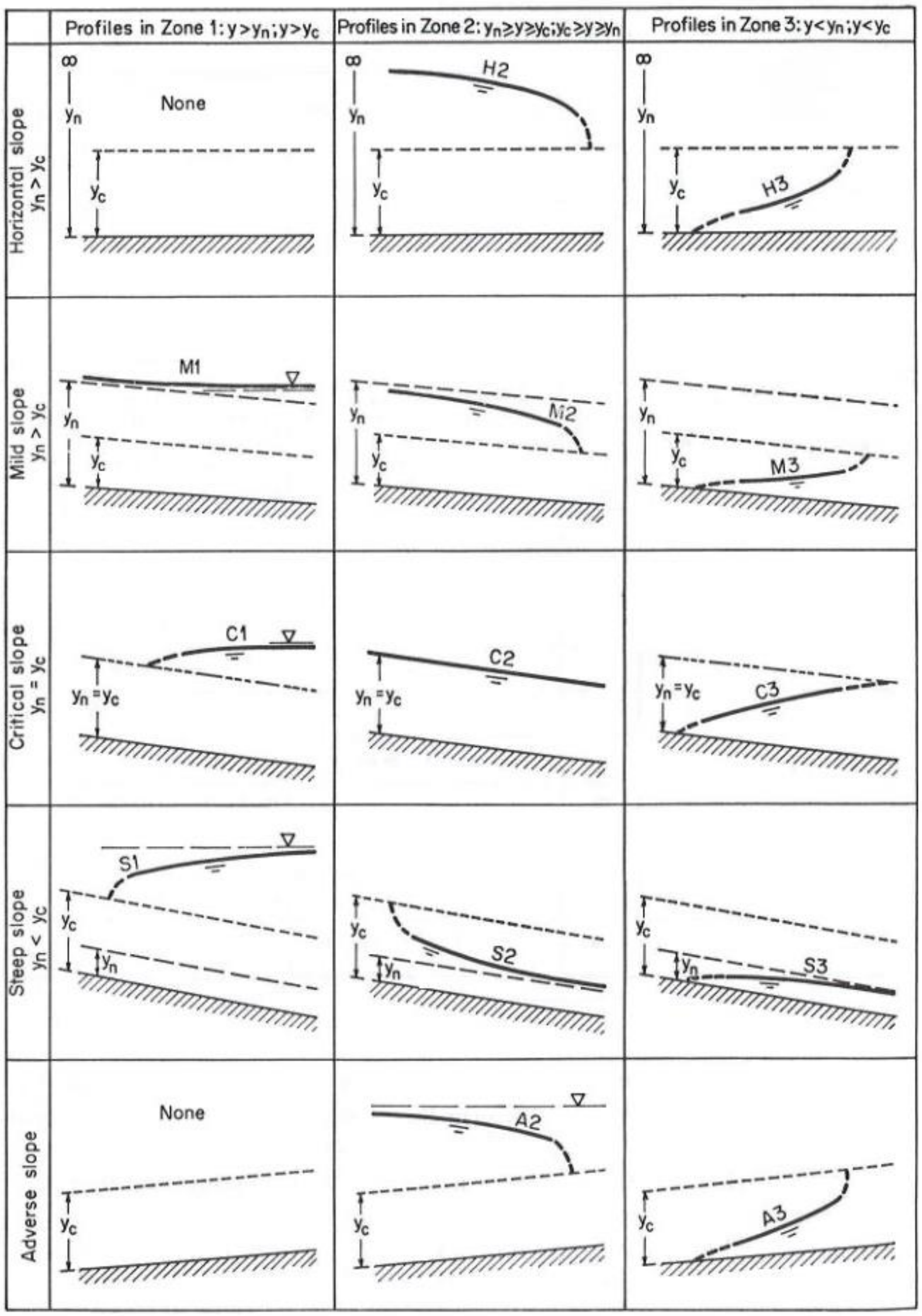
Chow (1959) describes the classification of gradually varied water surface profiles into fifteen different types. The profile types are based on five slope inclination categories and three profile zones. Each type reflects the location of the water surface profile relative to normal and critical depth channel flow conditions. These water surface profile types are designated according to an alphanumeric protocol, as follows and as shown in Figure 4.16. The profile types are designated as H2, H3; M1, M2, M3; C1, C3; S1, S2, S3; and A2, A3.

- The **letter** over the water surface profile describes the slope, i.e., **H** for horizontal, **M** for mild, **C** for critical, **S** for steep (supercritical), and **A** for adverse slope.
- The **number** over the water surface profile represents the zone number, where:
  - **Zone 1** – water surface above both normal and critical depths
  - **Zone 2 –** water surface between normal and critical depths

- **Zone 3 –** water surface below both normal and critical depths

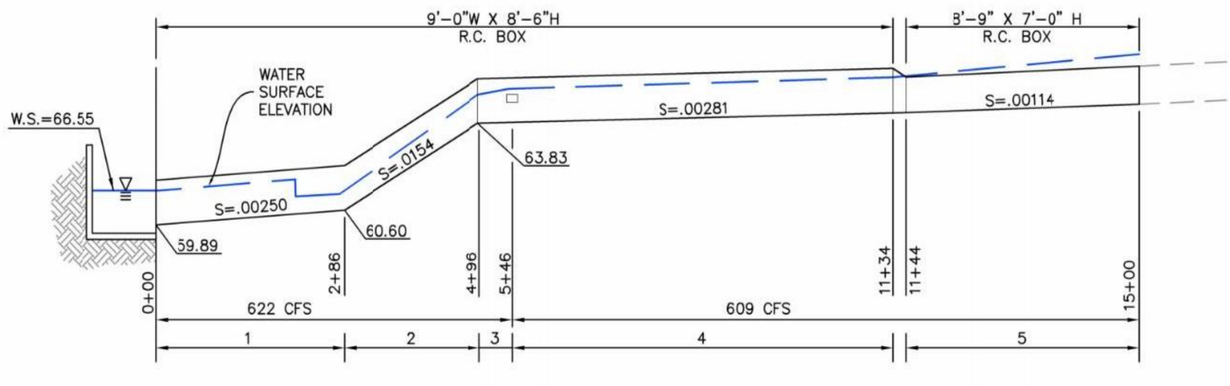
Flow profile analysis enables the designer to predict the general shape of the flow profile for a given channel layout. This step is a significant part of the open channel design process, and it should not be omitted. Flow profile analysis will serve to identify control sections and to provide a work plan for more detailed design calculations.

Figure 4.16: Classification of Flow Portion of Gradually Varied Flow
(Chow, 1959)



4.4.3.3 Example Problem

Figure 4.17: Example Problem Results



As discussed in Section 4.4.3.1, control points are locations in a channel where a relationship of depth, y, versus flowrate (Q) is known.

Table 4.11: Example Problem Results Summary

Reach	Flow Regime	Control Station	Control Point Depth
1	Subcritical	0+00	Known downstream WSE = 66.55
2	Supercritical	4+96	Grade break – critical depth elev =63.83+5.30 = 69.13
3	Subcritical	4+96	Grade break – critical depth elev = 69.13
4	Subcritical	5+46	Calculated from Junction Analysis based on calculations initiated in Reach 3
5	Subcritical	11+34	Calculated from Transition Analysis based on calculations initiated in Reach 4

- In Reach 1, Yn is greater than Yc (subcritical, so the profile calculation extend in the upstream direction. The downstream controlling water surface of 66.55ft will be used.

- In Reach 2, Yn is less than Yc, so flow in Reach 2 will be supercritical. Calculations will begin at the control point at Station 4+96 and extend downstream with an S2 curve. Somewhere between Reach 1 and 2 a hydraulic jump (rapidly varied flow) will occur to transition the shallow supercritical flow in reach 2 to meet the deeper subcritical flow in reach 1. See Section 4.3.5 for more information on calculating and locating hydraulic jumps, as well as the example problem in Section 4.4.5.1.

- In Reach 3, Yn is greater than Yc (subcritical), and therefore the profile calculation will proceed upstream from the control point (critical depth) at Station 4+96. Reach 3 will flow as a gradually varied 'M2' curve.

- At Station 5+46, there is a junction structure, which is analyzed as rapidly varied flow, as described in Section 4.3.2. Once the junction is analyzed, gradually varied flow calculations can proceed upstream as described below for Reach 4.

- In Reach 4, Yn is still greater than Yc (subcritical), and therefore the profile calculation will continue upstream beginning from the calculated water surface elevation upstream of the junction.

- Between Reach 4 and Reach 5 is a transition structure, which will be analyzed as described in Section 4.3.3.
- Reach 5 is still subcritical and therefore will be analyzed beginning at the upstream end of the transition structure and continue upstream to the end of Reach 5.

4.4.4 Step 4: Calculation of Gradually Varied Flow

Gradually Varied Water surface profiles are calculated using following formulas:

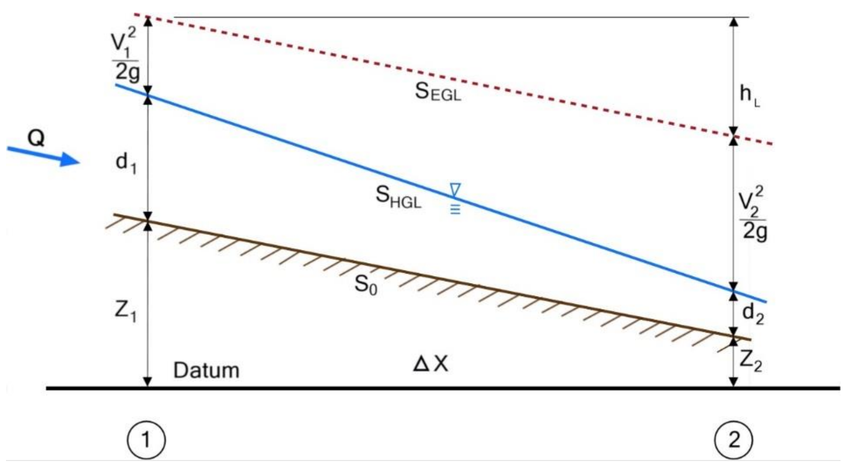
Formula	Reference
Conservation of Energy	See Section 4.2.3
Energy Losses	See Section 4.3

Calculations always begin at a location of known depth (control point) and extend upstream or downstream as follows:

Flow Regime	Direction of Calculation
Subcritical	Downstream
Supercritical	Upstream

There are two computational approaches that can be used to establish the water surface profile: Direct Step and Standard Step. A comparison of the two approaches can be found below.

[Note the graphic shows a supercritical example, where computations extend in the downstream direction, starting at location (1).]



	Direct Step	Standard Step
Given	d_1 and d_2	d_1 and ΔX
Find (Step Calculation)	ΔX	d_2
Pros	Easy to calculate	Allows determination of depth at desired locations.

Cons	• The channel cross section, slope, and material must be unchanged throughout the calculated length of ΔX, otherwise the result is invalid. • Doesn't easily allow determination of depth at specific desired locations	• Requires iterative calculations

Important Note:

In both methods, it is important to understand that the friction slope at location (1) will be different than the friction slope at location (2), and therefore the calculations will use the average of the two friction slopes (S*f-avg*). Because of this, it is also important to limit the 'step' distance. If the 'step' (either ΔX or Δd) is too large, the average friction slope will not be an accurate representation of the friction losses between the two locations. It is common practice to ensure that the change in EGL is less than 10% of the starting EGL.

4.4.4.1 Direct Step Method

The most straightforward technique to use in numerically solving the varied flow equation is referred to as the direct step method. The energy equation is rewritten and rearranged to isolate Δx. The distance between two sections with known depths is computed using Equation (4.54).

$$S_0 \Delta x + y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + S_f \Delta x \tag{4.51}$$

$$S_f = (S_{f1} + S_{f2})/2 = \text{Average slope of Energy Gradient} \tag{4.52}$$

$$S_o \Delta x + E_1 = E_2 + S_f \Delta x \tag{4.53}$$

$$\Delta x = \frac{(E_2 - E_1)}{(S_0 - S_f)} \tag{4.54}$$

Computational Procedure:

1) Determine a point in the channel where the depth is known, typically at a control point or previously calculated depth.
2) Assume a depth which is somewhat larger (for subcritical) or smaller (for supercritical) than the initial depth.
3) Compute the distance upstream or downstream to reach the chosen depth using Equation (4.54).
4) Repeat the process until the end of the reach or until normal depth is achieved.

4.4.4.2 Standard Step Method

The standard step method is calculated by assuming a flow depth at the chosen station where the flow depth is to be determined. It requires an iterative approach that is best suited to spreadsheets or computer software such as HEC-RAS. The energy gradient at this station is calculated by two independent procedures as follows:

1) Determine a point in the channel where the depth is known, typically at a control point or previously calculated depth.
2) Identify the desired station for your calculation and *estimate* a depth for that location. The estimated depth must be higher than your control point depth if the reach is subcritical, and lower if it is supercritical. The step procedure is carried out in downstream direction for supercritical flows and in the upstream direction for subcritical flows.
3) Calculate EGL2 based on both of the methods below:
   a. Calculate EGL_2 based on the assumed d_2. $EGL_2 = Z_2+d_2+V_2^2/2G$
   b. Estimate EGL_2 based on the losses that occur over the ΔX reach using the methods described in Section 4.3. $EGL_2 = EGL_1$ - Losses
4) If EGL_2 from step 3a doesn't match the EGL_2 from step 3b, choose an alternative assumption for d_2, until the two methods result in an EGL_2 that matches within 0.1 feet or less. If using a spreadsheet such as Microsoft Excel, the 'goal seek' function can automate this iteration for you.

4.4.4.3 Special Considerations for Gradually Varied Flow

- Superelevation around curves (See Section 8.3.9)
- Slug Flow and roll waves (See Section 8.3.8)
- Freeboard (see Section 6.2.1.2 and 8.3.7)

4.4.5 Step 5: Calculate Rapidly Varied Flow

Rapidly varied flow is characterized by very pronounced changes in the water surface profile over a short distance. The change in water surface profile may become so abrupt to result in a state of high turbulence. Calculation methods for gradually-varying flow (e.g., direct-step and standard-step methods) do not apply for rapidly-varying flow. There are mathematical solutions to some specific cases of rapidly varying flow, but the solutions to most rapidly varying flow problems rely on empirical data.

Each of these flow conditions require detailed calculations to properly identify the flow capacities and depths of flow in the given section. The design engineer must be cognizant of the design requirements for rapidly varying flow conditions and shall include all necessary calculations as part of the design submittal documents. This manual provides methods and guidance for three types of rapidly varied flow including Junctions (4.3.2), Hydraulic Jumps (4.3.5), and Piers (4.3.6). For additional resources or scenarios not covered in this manual, the design engineer is referred to the hydraulic references in Section 4.5 for the proper calculation methods to use in the design of drainage facilities with rapidly varying flow conditions.

4.4.5.1 Example Problem #1: Hydraulic Jump

Example Problem: Hydraulic Jump in RCB

RCB, width = 9 feet, n = 0.014 and Q= 622 cfs. Invert longitudinal slope changes from 0.0154 (upstream reach) to 0.0025 (downstream reach). For this example, assume that the upstream and downstream channels are infinitely long, meaning that the water surface approaching the grade break is at normal depth.

Determine if a hydraulic jump occurs. If jump does occur, determine sequent depth, jump length, and locate position of jump.

Solution:

1) From Table 4.10, normal depth, critical depth and the associated flow regimes were previously calculated for reaches 1 and 2.

Reach	Section	Q (cfs)	$y_c = \left(\frac{Q^2}{b^2 g}\right)^{1/3}$	$K' = \frac{nQ}{b^{8/3} S^{1/2}}$	Yn/b Table 7-11 King's	Yn (ft)	Flow Regime
1	9'-0"W x 8'-6"H	622	5.30	0.498	0.75	6.75	Subcritical
2	9'-0"W x 8'-6"H	622	5.30	0.201	0.38	3.42	Supercritical

Since flow changes from supercritical in Reach 2 to subcritical flow in Reach 1, a jump can occur.

2) **Sequent Depth**: Use Equation (4.46) to calculate jump height (sequent depth).

$$Y_2 = \frac{1}{2} Y_1 \left[\sqrt{1 + 8F_{r1}^2} - 1\right] \tag{4.46}$$

Simplifying Equation (4.46) for a rectangular section

$$Y_2 = -\frac{1}{2} Y_1 + \left(\frac{Y_1^2}{4} + \frac{2V_1^2 Y_1}{g}\right)^{1/2}$$

$$Y_2 = -\frac{3.42}{2} + \left(\frac{3.42^2}{4} + \frac{2(20.2)^2(3.42)}{32.2}\right)^{1/2} = 7.8ft$$

3) **Jump Length**: Use Figure 4.11 to determine Jump Length in terms of sequent depth. For the x-axis determine the Froude number for the supercritical reach.

$$F_1 = \frac{V_1}{\sqrt{g\, y_1}} = \frac{20.2}{\sqrt{(32.2)(3.42)}} = 1.92$$

From Figure 4.11, when $F_1 = 1.92$, $L/y_2 = 4.2$
Therefore, jump length L = (4.2)(7.8) = 32.8'

4) **Jump Location**: Calculate (P + M) using Equation (4.48) at grade break to locate position of jump upstream or downstream of that point.
   a. Steep Slope Supercritical ($S_0 = 0.0154$)
   $$P + M = A\, y_C + \frac{Q^2}{(g\, A)} = 9'(3.42)(5.3) + \frac{622^2}{32.2\,(3.42)(9)} = 553.5 \text{ ft}$$
   b. Flat Slope Subcritical ($S_0 = 0.0025$)

$$P + M = A\,y_C + \frac{Q^2}{(g\,A)} = 9'(6.75)(5.3) + \frac{622^2}{32.2\,(6.75)(9')} = \text{ 519.8 ft}$$

c. Comparison of (P + M) at grade break

[(P + M) sub = 519.8'] < [(P + M)super = 553.5']
Therefore, jump occurs upstream of break on subcritical slope (S_0 = 0.0025)

4.4.5.2 Example Problem #2: Junction Analysis

At Station 5+46, inflow occurs and a junction analysis must be performed in order to determine the control for Reach 4. This depth is calculated as follows:

$$P_2 + M_2 = P_1 + M_1 + M_3 \cos\theta \tag{4.55}$$

$$\frac{b\,D_2^2}{2} + \frac{Q_2^2}{bD_2 g} = \frac{bD_1^2}{2} + \frac{Q_1^2}{bD_1 g} + \frac{Q_3^2}{A_3 g}\cos\theta \tag{4.56}$$

$$\frac{(9)\,(5.8)^2}{2} + \frac{(622)^2}{(9)(5.8)(32.2)} = \frac{(9)D_1^2}{2} + \frac{(609)^2}{(9)D_1(32.2)} + \frac{(13)^2(0.707)}{(6.28)(32.2)} \tag{4.56}$$

$$151.4 + 230.2 = 4.5D_1^2 + \frac{1279.8}{D_1} + 0.6$$

$$4.5D_1^2 - 381.0D_1 + 1279.8 = 0$$

By trial and error, D_1 = 6.2 feet.

Tabled values of Dn and Dc indicate that the profile calculation should proceed upstream in Reaches 4 and 5. Since flow in the conduit becomes sealed somewhere in the curve in Reach 5, pressure flow is assumed for the length of the curve and the remainder of the reach, in lieu of a superelevated open channel water surface.

In Reach 2, Dn is less than Dc, and therefore the profile calculation should proceed downstream from the control point at Station 4+96. When critical depth is reached at Station 0+87, calculations are initiated at the next downstream control point which is the outlet. In Reach 1, Dn is greater than Dc, and therefore the profile calculation should proceed upstream until critical depth is reached at Station 3+23. Between Stations 0+87 and 3+23, there are two alternative stages of flow and the necessary conditions to produce a hydraulic jump. The exact location of the jump is usually not required but can be determined by equating pressure plus momentum for upper and lower stages.

4.4.6 Allowable Software

The District has identified software that provides results that are consistent with the methods described in this chapter. Please refer to District Accepted Software.

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5 STREET DRAINAGE

5.1 INTRODUCTION

5.1.1 General Discussion

The intent of this chapter is to provide criteria and methodologies for design and evaluation of surface drainage systems consisting of street drainage and storm drain catch basin (inlets). In many cases roadways act as the primary means of conveying surface water runoff to storm drain systems. Limiting the depth and velocity of runoff within the roadways allows traffic to move safely and efficiently. The combined effect of the street and storm drain conveyance systems should be to provide 100-year protection for habitable structures. Where feasible, pollutant control water quality BMPs may be incorporated into the surface drainage system. Depending on project location, local agency may require Engineer to implement trash capture devices.

The following methods described are to be applicable for sizing all of the District's current Standard Catch Basins, which are as follows: Catch Basin No. 1 (CB100), Catch Basin No. 4 (CB101), and Catch Basin No. 6 (CB102), as well as the following Riverside County Transportation Department Standards: 300, 301, 302, 304, 305, 311, and 312. Note the following methods do not apply for the Concrete Drop Inlet (CB110).

5.1.2 Source of Data

This chapter describes methodology that should be used for the estimation of street flow capacity, allowable spread, and catch basin design. The procedures, equations, and nomographs in this Section are adapted from Chapter 4, Section 4 of the Federal Highway Administration, Hydraulic Engineering Circular No. 22 (HEC-22) (not to be confused with the Army Corps' *Hydrologic Engineering Center*), *Urban Drainage Design Manual* (USDOT, FHWA, 2009), and U.S. Department of Transportation, Federal Highway Administration (FHWA), March 1984, *Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements*. Nomographs presented in this chapter can be used for design concept evaluations or initial evaluations. The appropriate sizing equations should be selected with deliberate consideration given to the constraints specified within HEC-22. Provided that the user has an understanding of those constraints, the District accepts the following software implementations of HEC-22:

- Bentley FlowMaster (for curb Opening and Combination Inlets)
- CIVILD Program Package – General Hydraulics Version 6.0 (for Curb Opening Inlets only)
- Advanced Engineering Software (AES) Hele1/2
- Manual Calculations

5.2 PROCEDURES

5.2.1 General Considerations

The procedures herein are established for the collection and conveyance of storm drainage within roadways.

New Road Drainage Capacity: Typical street sections can be obtained from the Riverside County Transportation Department, or for projects within an incorporated city, the appropriate local governmental agency. The capacity of newly designed surface drainage systems (street capacity and inlet interception capacity) must be consistent with that of the downstream stormwater conveyance system (discussed in Chapter 6). The roadway conveyance capacity is constrained by the width, depth, and velocity of flow while accounting for variations in road profile and cross sections at features such as sumps and intersections.

Existing Road Drainage Capacity: For existing roads, the street capacity shall be determined by analyzing actual surveyed street cross sections at critical capacity-limited locations. This actual capacity may not match the capacity of a "typical" section pursuant to County Road standards.

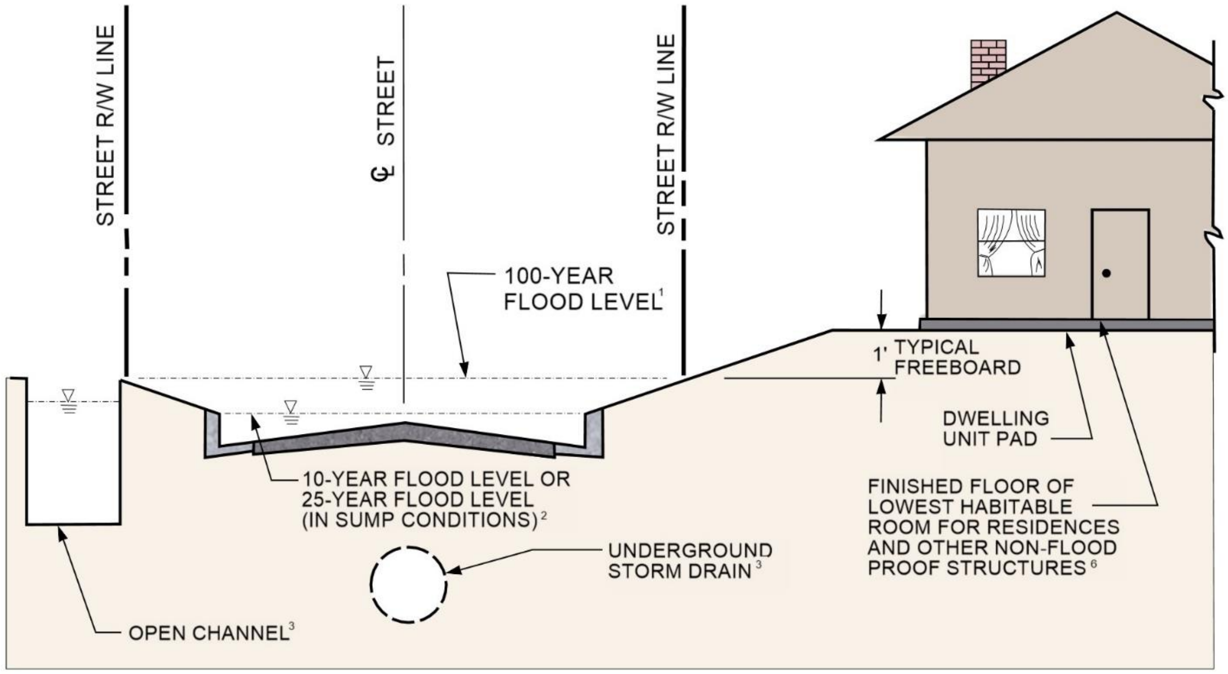
Catch Basin Inlets: Inlet interception capacity is constrained by surface runoff approach velocity and depth, inlet geometry, available ponding head, as determined from street capacity calculations, and in some cases backwater constraints from the downstream underground storm drain system.

Storm drain inlets must be placed at prescribed locations to protect the public safety, manage flooding and nuisance flows, and prolong pavement life. Inlets shall be placed to meet the following criteria:

- Maintain 100-year flow depth to the ground surface elevation at the street right of way;
- Maintain 10-year flow depth at top of curb elevation;
- Locate within sumps in the roadway profile;
- Locate where flow is directed toward an intersection that does not have a cross gutter;
- Locate in places where super-elevation occurs on arterial highways, an inlet shall be provided at the median, if applicable, or provided (up grade of the point water starts to move across the lanes) as necessary to preclude drainage across the travel lanes; and
- Locate at the downstream end of system to prevent bypass flows and capture surface runoff into the system.

It is recommended for safety reasons that inlets are placed to maintain maximum value of the product of the flow velocity, v (fps), times the depth of water at the curb, y (ft), is less than or equal to 6.

Figure 5.1: Flood Protection Criteria



Notes:

1- The 100-year flood level shall be contained within street right of way limits.
2- The 10-year flood level or 25-year flood level (in sump conditions) shall be contained within the top of curbs.
3- Initiate a storm drain or channel when either condition is exceeded.
4- Protection criteria shown are the Districts typical minimum requirements. Special conditions or other authorities may require stricter control; i.e., for reasons of traffic or pedestrian safety, maintenance problems behind curbs, etc., lower maximum depths of flow in street may be required. Also see Riverside Co. Ord. No. 460.
5- For arterial highway, collector street, and local street, depth (y) times velocity (v) should not exceed 6 for safety reasons.
6- For County regulated floodplains, the elevation of the lowest floor of the building, including basement, or cellars must be at least 1 foot above the Base Flood Elevation (BFE).

5.2.1.1 Catch Basin Types

Catch basins used for drainage can be divided into four main categories, curb-opening catch basins, grated catch basins, combination catch basins (curb-opening plus grate), and slotted drain catch basins. Typical catch basin inlets are shown in Figure 5.2. Catch basins may be further classified as being on-grade or in a sump. The on-grade condition exists when the gutter profile falls across the length of the catch basin. The sump condition exists at the low point in the gutter profile where ponding occurs, and water is restricted from bypassing the catch basin area. This may be due to a change in grade of the street from positive to negative or due to the crown slope of a cross street when the catch basin is located at an intersection.

Curb-opening catch basins are typically preferred as they are not easily prone to clogging and offer little interference to pedestrian, bicycle, and vehicular traffic. A depressed-curb opening inlet is hydraulically more efficient than an undepressed curb-opening inlet.

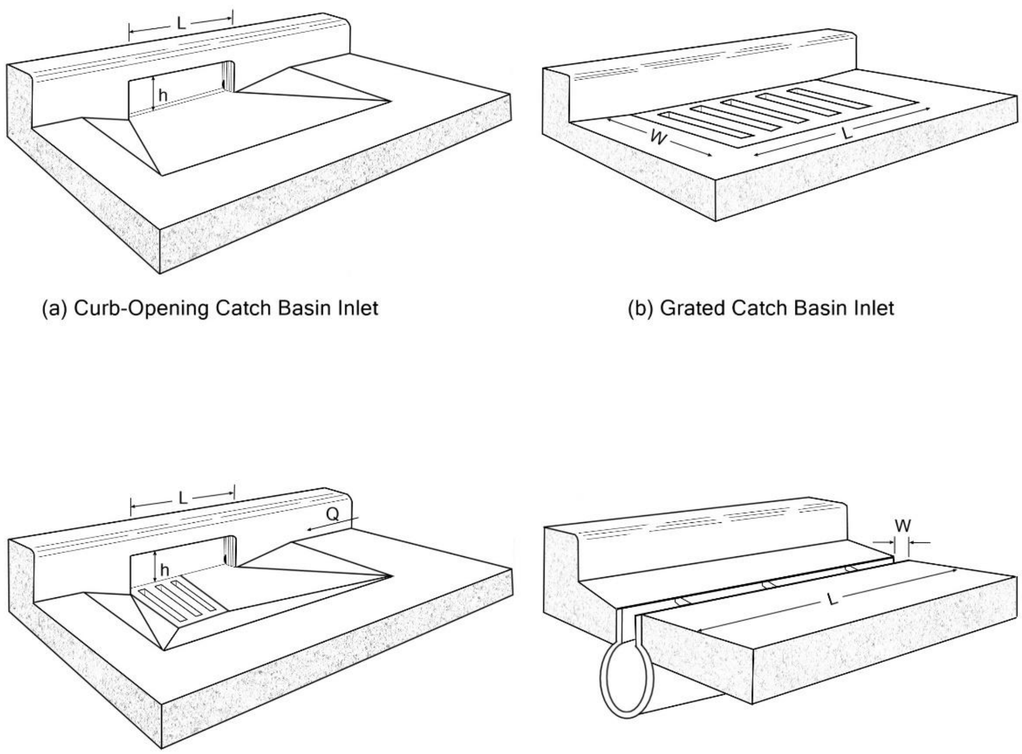
Grated catch basins are typically rectangular openings in the gutter covered by one or more traffic rated grates. Grated catch basins may be depressed or undepressed. When optimizing for hydraulic efficiency, the engineer shall also consider bicycle and pedestrian safety (e.g., bicycle proof grate) and structural adequacy.

Combination type catch basin inlets are characterized by use of a curb-opening and grated catch basins acting together as one unit. A combination catch basin typically has a curb-opening longer than and placed upstream of the grated catch basin inlet. This allows the curb-opening, occasionally referred to as a "sweeper", to intercept debris which might otherwise clog the grate. A combination inlet with a curb-opening adjacent to a grate has an interception capacity equal to the sum of the curb opening capacity (upstream of the grate only) and the grate capacity (ignoring the curb opening immediately adjacent to the grate).

In a sump, combination inlets may be desirable if additional capacity beyond that of the curb opening is needed.

A slotted drain is a narrow opening in the pavement which intercepts sheet flow and conveys it through a pipe (normally corrugated steel). Slotted drains are most effective when street slopes are shallow. Slotted drains can be used on curbed or uncurbed roadways. The design of slotted drains is outside the scope of this manual and these drains shall only be specified with prior agency approval.

Figure 5.2: Catch Basin Inlets



(a) Curb-Opening Catch Basin Inlet

(b) Grated Catch Basin Inlet

(c) Combination Catch Basin Inlet

(d) Slotted Catch Basin Inlet

5.2.2 Street Capacity

When estimating the total capacity of a roadway (curb to curb or sidewalk to sidewalk), there are two options: 1) using Manning's equation via hand calculations, spreadsheets, WSPG, Flowmaster, etc., or 2) HEC-22 calculations for spread presented in Section 5.2.2.1. Manning's equation for uniform flow (ignoring the friction along the vertical face as insignificant) as expressed in Equation (5.1) may be used. Values calculated are for a street half width. Once the flow exceeds the crown elevation, a split flow condition may exist. Design Example 2 shows how to address a split flow condition.

Per County of Riverside Transportation Department Improvement Plan Check Policies and Guidelines, Section K Drainage Plans, note 7, street capacity checks shall be performed to ensure that all ultimate road improvements comply with the drainage requirements in Section 11.1 of Ordinance No. 460 for Subdivisions or Policy 3.18 of the General Plan Safety Element and the Riverside County Hydrology and Hydraulics Manuals for all other projects. The 10-year frequency design discharge shall be contained between the tops of curbs or asphalt concrete dikes, and the 100-year frequency design discharge shall be contained within the street right of way. If either of these conditions is exceeded or those described in Section 5.2.1, additional flood control facilities shall be provided.

Generally, road superelevation must be approved by the Transportation Department or approved with the entitlement for the project. The County may, at its sole discretion, require storm drain or other flood control facilities for future use. Where superelevation in combination with raised median is proposed, the 10-year event for the higher street portion would need to be contained within the top of the raised median. If the project is only building the lower portion of a superelevated road, the 10-year event would need to be contained within the top of the lower curb or asphalt dike, without considering the capacity for a future median, or otherwise approved by the Transportation Department.

For projects building the higher portion of a superelevated road with a raised median or if no median is proposed, a separate analysis of the 10-year ultimate flow width would be required. The ultimate catch basins on the lower end may be required if the right of way exists on the opposite (lower) side of the road. Otherwise, sufficient interim inlets or down-drains may be required by the Transportation Department.

$$Q = \left(\frac{1.49}{n}\right) AR^{0.67} S^{0.5} \qquad (5.1)$$

where:

- Q = total flow (cfs)
- n = Manning's roughness coefficient. A n-value of 0.015 is typically used for paved streets unless special conditions exist
- A = flow area (sq ft)
- R = hydraulic radius (ft)
- S = slope of energy grade line, assumed equal to longitudinal street gutter slope (ft/ft) not roadway cross fall

Figure 5.3: Riverside County Standard A-6 and A-8 Curb and Gutter

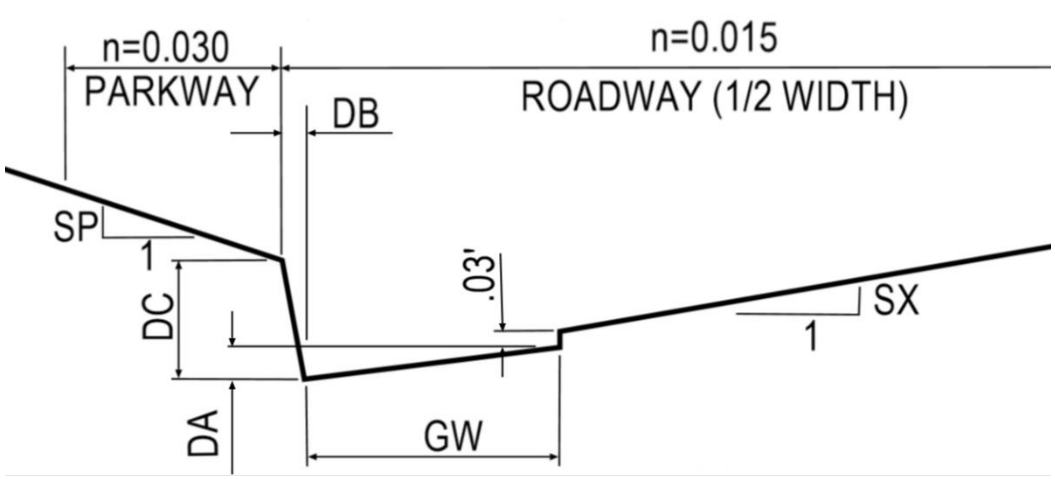


Table 5.1: Standard A-6 and A-8 Curb (Std 200 and 201) Dimensions

Curb Type	DA	DB	DC	GW	SP	SX
A-6	0.125’	0.125’	0.5’	2’	0.02	0.02
A-8	0.125’	0.1667’	0.67’	2’	0.02	0.02

where:

DA = gutter hike (ft)
DB = curb face batter (ft)
DC = curb height (ft)
GW = gutter width (ft)
SP = slope of parkway (ft/ft)
SX (S_x) = street pavement cross (transverse) slope (ft/ft)

5.2.2.1 Dry Lane Requirements

When the allowable pavement spread has been determined based on curb height or dry lane requirements per local jurisdiction, the gutter plus street carrying capacity shall be computed using the modified Manning’s formula as expressed in Equation (5.2). This equation is no longer valid once flow exceeds the half-street to crown capacity. A dry lane is where one travel lane is kept free from inundation during a certain design storm event.

$$Q_t = \left(\frac{0.56}{n}\right) S_x^{1.67} S^{0.5} T^{2.67} \tag{5.2}$$

where:

Q_t = theoretical gutter plus street carrying capacity (cfs)
n = Manning’s roughness coefficient. A n-value of 0.015 is typically used for paved streets unless special conditions exist
S_x = street pavement cross (transverse) slope (ft/ft)
S = longitudinal slopes (ft/ft)
T = spread of flow on pavement (ft)

The relationship between the spread of flow and depth is expressed in Equation (5.3).

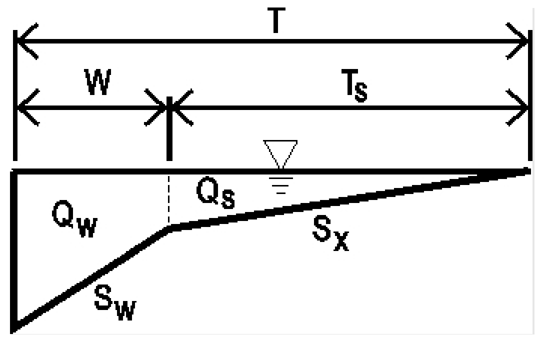
$$T = d/S_x \tag{5.3}$$

where:

T = spread of flow on pavement (ft)
d = depth of flow (ft)
S_x = pavement cross slope (ft/ft)

For gutters and pavement street section with composite cross-slopes ($S_W \neq S_x$), pavement spread is determined using the relationships presented in Figure 5.4.

Figure 5.4: Composite Cross-Slope of Gutter and Street Section



To determine discharge in a street (pavement plus gutter) with a composite cross-slope, a multi-step analysis is required. First, find Q_S using Equation (5.4). Next, find the total street flow (Q) using Equation (5.6) or Figure 5.5. Then determine the ratio of flow in the depressed section to total street flow using Equation (5.5). Gutter flow (Q_W) can then be determined using Equation (5.7).

$$Q_s = \left(\frac{0.56}{n}\right) S_x^{1.67} S^{0.5} T_s^{2.67} \tag{5.4}$$

where:

Q_S = flow rate in paved area (cfs), excluding the gutter
n = Manning's roughness coefficient. A n-value of 0.015 is typically used for paved streets unless special conditions exist
S_x = pavement cross slope (ft/ft)
S = roadway longitudinal slope (ft/ft)
T_S = spread of flow on pavement for a composite section (ft)

$$E_o = 1 \Bigg/ \left(1 + \frac{S_W/S_x}{\left[1+\frac{S_W/S_x}{\frac{T}{W}-1}\right]^{2.67} - 1}\right) \tag{5.5}$$

where:

E_O = ratio of flow in the depressed section to total street flow
S_x = pavement cross slope (ft/ft)
W = width of gutter (ft)
T = width of flow, spread (ft)
S_W = cross slope of depressed gutter $\left(S_x + \frac{a}{W}\right)$ (ft/ft)
a = gutter depression, see Figure 5.3, (ft)

(Equation (5.4), Reference: USDOT, FHWA, 2009, HEC-22, Equation 4-4)

$$Q = \frac{Q_S}{(1-E_o)} \tag{5.6}$$

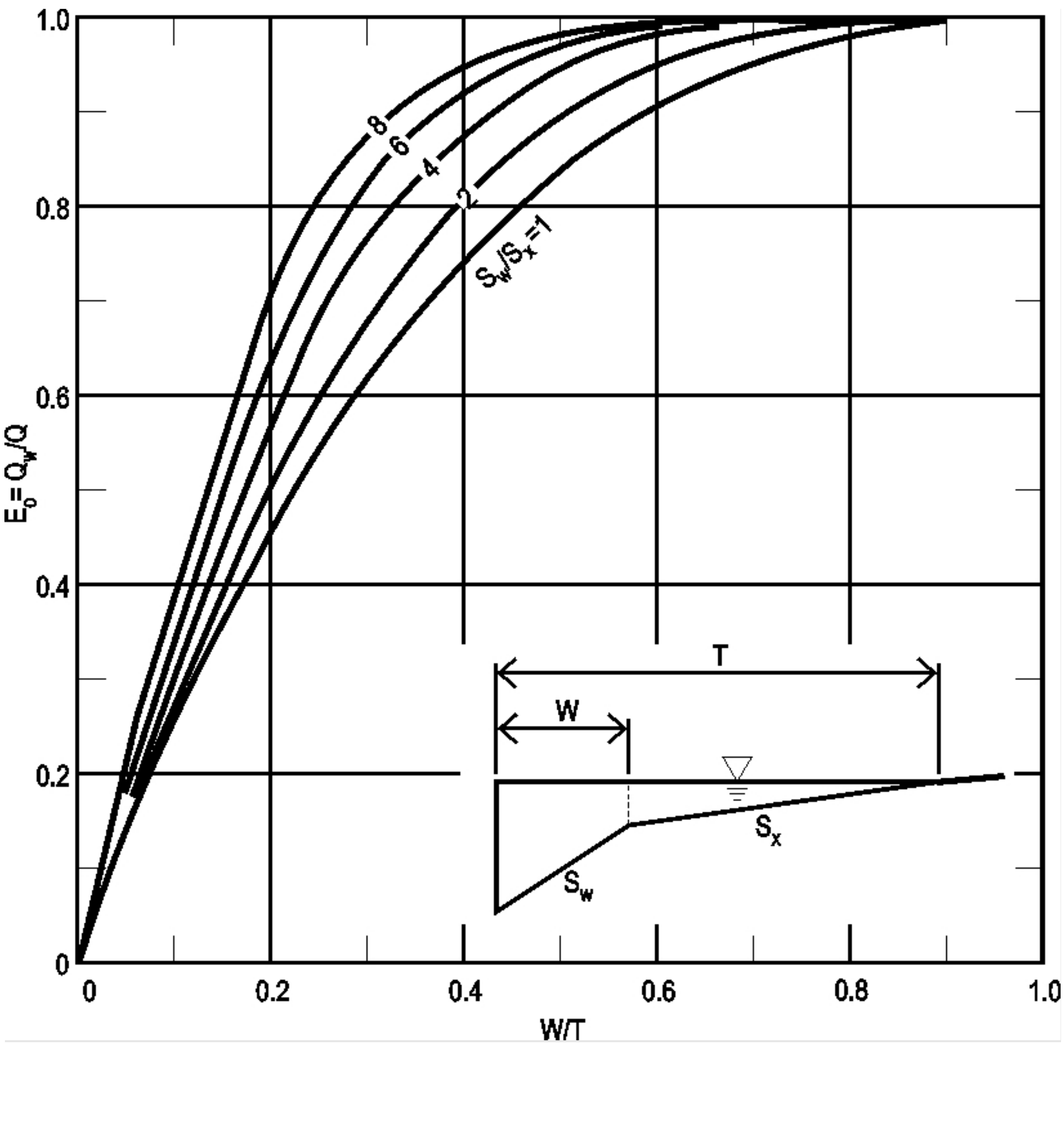
$$Q_w = Q - Q_s \tag{5.7}$$

where:

Q_W	=	flow rate in depressed section of gutter (cfs)
Q	=	total gutter plus pavement flow rate (cfs) at flow width T (ft)
Q_S	=	flow rate in paved area (cfs)

Figure 5.5: Ratio of Frontal Flow to Total Gutter Flow

(USDOT, FHWA, 1984, HEC-12, CHART 4)



5.2.2.2 Cross Gutters

For drainage purposes, conventional cross gutters may be used perpendicular to the local street to transport runoff across local street intersections when approved by the governmental agency.

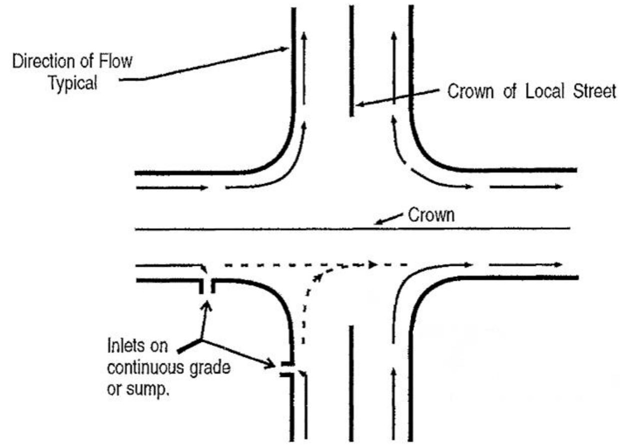
The cross gutter should be sufficient to transport the runoff across the intersection with a spread equivalent to that allowed on the street, such as when a dry lane is required.

The carrying capacity of each gutter approaching an intersection shall be calculated based upon the effective slope, as outlined herein.

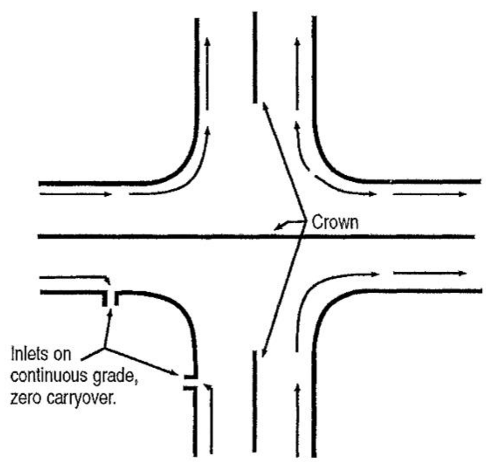
When the gutter slope will be continued across an intersection, as when cross gutters are in place, use the slope of the gutter flow line crossing the street to calculate capacity. When the gutter flow must undergo a direction change at the intersection greater than 45 degrees, the slope used for calculating capacity shall be the effective gutter slope, defined as the average of the gutter slopes at 0 feet, and 50 feet upstream from the point of direction change (BC of the curb return).

In the case where two arterial streets intersect, the longitudinal grade of the more major street shall be maintained as much as possible. No form of cross gutter shall be constructed across major streets for drainage purposes. Cross gutters across the intersection may be considered for collector streets, but only in rare cases and with agency approval.

Figure 5.6: Typical Street Intersection Drainage to Storm Drain System



Local or Collector to Arterial



Arterial to Arterial

(One Continuous Crown)

5.2.3 Curb-Opening Catch Basins

Standard Riverside County curb opening inlet lengths are 7’, 14’, 21’, and 28’, however, alternative lengths may be used. Curb-opening catch basins are based on District Standard Drawing CB100 and Riverside County Transportation Department Standard 300. Design engineers should confirm with the local jurisdiction for the applicable catch basin standard drawing.

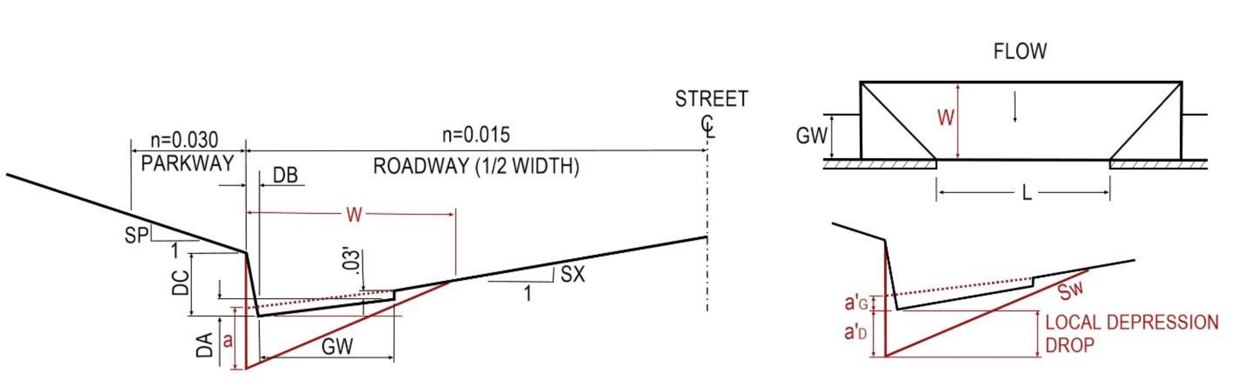
It is not always possible to intercept all street flow with a single inlet, and a portion of the approaching flow will continue past the inlet area as "bypass flow". The bypassed flow must be included in the capacity calculations for the next downstream inlet. In situations where a catch basin is receiving bypass from an upstream catch basin, the connector pipe for the catch basin receiving bypass flows must be designed for the tributary flows to that catch basin plus the bypass flows. The flow in the connector pipe for each catch basin is the flow intercepted by the catch basin in a 100-year event.

$$Q_{Bypass} = Q_{Total\ Approaching} - Q_{Intercepted\ by\ C.B.}$$

If the required calculated catch basin length is greater than 28', a combination of multiple catch basin sizes must then be used. In the occurrence of sequential catch basins, continuous local depression may be used as shown on the Riverside County Transportation Department Standard No. 311 (Case "A"); however, continuous local depressions may not be allowed depending on local agency criteria.

For all catch basin inlets, hydraulic analyses may be calculated by using: 1) accepted software, or 2) follow HEC-22 criteria with hand calculations. See examples using both hand calculations and software in Section 5.3.

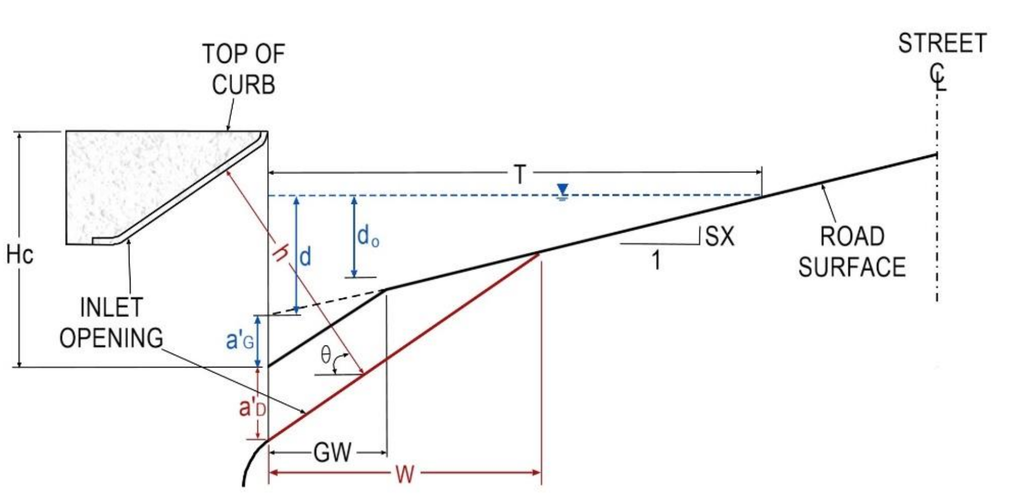
Figure 5.7: Riverside County Local Depression



where:

a'_G	=	gutter depression compared to pavement cross slope projection to curb (ft) = 0.116' typically
a'_D	=	local depression drop at catch basin (ft) = 0.25' typically
a	=	gutter depression = $a'_G + a'_D$ = 0.45' typically
W	=	local depression width (ft) = 4' typically

Figure 5.8: Curb Opening Catch Basin Inlet Inclined Throat



5.2.3.1 *Curb Inlet Catch Basins On-Grade*

The capacity of a curb inlet on-grade depends on gutter longitudinal slope, depth of flow in the gutter, the dimensions of the curb opening, and the amount of depression at the catch basin. The standard curb opening inlet includes a depressed gutter section or local depression. The length (L_t) of curb opening catch basin required for total interception of gutter flow on a composite pavement section cross slope is expressed as:

$$L_{t=}\ 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \tag{5.8}$$

where:

L_t	=	curb opening length required to intercept 100% of gutter flow (ft)
Q	=	roadway half width total flow rate (cfs)
S	=	longitudinal slope (ft/ft), not cross slope
n	=	Manning's roughness coefficient
S_e	=	equivalent pavement cross-slope (ft/ft)

The equivalent cross slope, S_e can be computed using Equation (5.9).

$$S_e = S_x + S_w' E_o \tag{5.9}$$

where:

S_e	=	equivalent cross slope
S_X	=	pavement cross-slope (ft/ft)
S'_W	=	cross slope of the gutter or local depression (at the inlet) measured from the cross slope of the pavement (ft/ft) ($S'_w = a/W$ see Figure 5.3)
E_O	=	ratio of flow in the depressed gutter section to total pavement plus gutter flow. This value is zero if there is no gutter depression at the inlet. See Equation (5.5).

The efficiency (E) of curb-opening catch basins shorter than the length required for total interception is:

$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \tag{5.10}$$

where:

E	=	Efficiency of curb-opening catch basins shorter than length required for total interception
L	=	Length of clear opening (curb, grate, or slot) of proposed inlet (ft)
L_t	=	Curb opening length required to intercept 100% of the gutter flow (ft)

The intercepted and bypassed flows for an inlet on grade with a given length can be calculated using the efficiency E. The intercepted flow Qin = Q_E. The bypass flow Q_{BYPASS} = (1 - E)Q.

5.2.3.2 Curb Inlet Catch Basins in a Sump

The capacity of a curb-opening catch basin in a sag or sump depends on water depth at the curb (d), the curb opening length (L), and the height of the curb opening. For catch basins without a local depression, the catch basin inlet operates as a weir for depths of water up to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. Local depression depth is taken into consideration when evaluating the weir depth for depressed catch basins (see Equation (5.12) below). At water depths between 1.0 and 1.4 times the weir depth, flow is in a transition stage.

At the transition stage, the designer shall calculate the capacity of the inlet under each condition and adopt a design capacity equal to the smaller of the two results. When designing the size of a curb-opening catch basin, the designer shall use the larger of the sizes for a given flow rate obtained by solving for the two conditions.

The weir location for a depressed curb-opening catch basin is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb-opening.

Inlets located in sump locations must be able to intercept the peak discharge from a 100-year design event. **Inlets within sump locations require a secondary (emergency) outlet that is clear to the sky (i.e., a swale or ditch rather than a second catch basin)** unless approved otherwise by the local agency. The depth of water in the emergency outlet must be at least one foot below adjacent finished floor elevations in cases where the flow path is between dwelling units. The emergency outlet must also be placed within a drainage easement to ensure conveyance is maintained in perpetuity. This emergency outlet system must direct overflows to either a downstream street with adequate capacity, a natural conveyance system, or an alternative system acceptable to the local agency. The point of discharge must also be analyzed to prevent downstream impacts.

The weir equation for the interception capacity of a depressed curb opening-catch basin is:

$$Q_i = C_w(L + 1.8W)d^{1.5} \tag{5.11}$$

where:

Q_i	=	amount of street flow intercepted by inlet (cfs)
C_W	=	weir coefficient = 2.3 (or 3.0 when d<h and L<12ft)

L = length of curb opening (ft)
W = width of gutter depression (ft)
d = depth of flow (ft) measured from water surface to projected cross slope

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation (5.11) for a depressed curb opening catch basin is:

$$d < h + a'_D \tag{5.12}$$

where:
h = height of curb opening or throat width (ft)
a'_D = gutter depression within the local depression (feet)

The weir equation for curb opening catch basins without depression ($W = 0$) becomes:

$$Q_i = C_w L d^{1.5} \tag{5.13}$$

where:
Q_i = amount of street flow intercepted by inlet (cfs)
C_W = weir coefficient = 3.0
L = length of curb opening (ft)
d = depth of flow (ft)

The depth limitation for operation as a weir becomes: $d \leq h$

Curb opening catch basins operate as orifices at depths greater than approximately 1.4h. The interception capacity can be computed by Equation (5.14):

$$Q_i = C_o h L (2 g d_o)^{0.5} \tag{5.14}$$

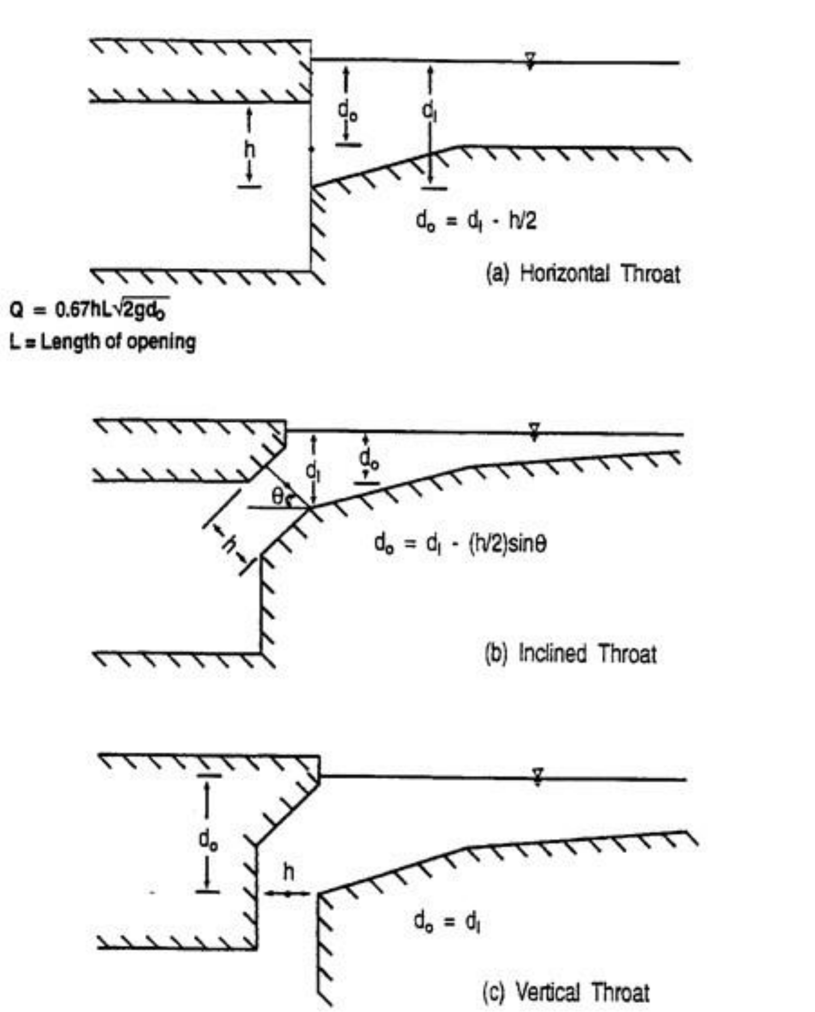
where:
Q_i = amount of street flow intercepted by inlet (cfs)
C_O = orifice coefficient = 0.67
h = height of curb opening catch basin, curb-opening orifice, or orifice throat (ft)
L = length of curb opening (ft)
g = acceleration due to gravity 32.2 ft/sec ^{2}
d_O = effective depth at the center of the curb opening orifice (ft), see Equation (5.15)

Equation (5.14) is applicable to depressed and undepressed curb opening catch basins and the depth at the opening includes any gutter depression.

Height of the orifice in Equation (5.14) assumes a vertical orifice opening (horizontal throat). As illustrated in Figure 5.9, other orifice throat orientations can change the effective depth on the orifice and the dimension $((d + a'_G + a'_D) - h/2)$. A limited throat width could reduce the capacity of the catch basin by causing it to go into orifice flow at depths less than the height of the opening.

Figure 5.9 : Curb Opening Catch Basin Inlets in Sump Condition

(Modified from: USDOT, FHWA, 1984, HEC-12, Figure 21)



The effective depth of flow at the curb face includes the curb depression and must be adjusted for the curb inlet throat configuration. Riverside County Standard curb inlet opening (District's Standard CB105 and TLMA's Standard 304) has an **inclined throat**, and therefore the effective depth of flow at the curb face is given by the expression:

$$d_o = (d + a'_G + a'_D) - \frac{h}{2}\sin\theta \qquad (5.15)$$

where:

d_o	=	effective depth of flow at the curb face (ft)
$d+a'_G$	=	depth of flow in adjacent gutter (ft)
a'_D	=	curb inlet depression (ft)
h	=	height of inclined throat curb inlet; h=[(h'c+a'D)/cos(24.5°)]-β, where a'D=4" see Table 5.2, h'c = curb height
β	=	curb batter type, Riverside County uses 1:4 batter, see Table 5.2
θ	=	throat incline angle is 56.31° degrees (valid for RCFC Standard Drawing CB105 and TLMA Standard Drawing No. 304); $\theta = 90° - \tan^{-1}(2/3) = 56.31°$

Table 5.2: Values of β and h based on Curb Batter Type

Height of Curb Face, h'c	Values of β Based on Curb Batter Type		Values of h for inclined throat inlet Based on Curb Batter Type	
	1:h to 4:v	1:h to 3:v	1:h to 4:v	1:h to 3:v
6" Curb Face	2.03"	1.68"	8.96"	9.31"
8" Curb Face	2.19"	1.78"	11.0"	11.41"
10" Curb Face	2.35"	1.88"	13.04"	13.51"
12 " Curb Face	2.51"	1.97"	15.07"	15.61"

5.2.4 Grated Catch Basins

The District's standard grated catch basin is per District Standard Drawing CB101. Design engineers should confirm with local jurisdiction for the applicable catch basin standard drawing.

For all catch basin inlets, hydraulic analyses may be calculated by using: 1) accepted software, or 2) follow HEC-22 criteria with hand calculations. See examples using both hand calculations and software in Section 5.3. It should be noted that the software has built-in grate types that may not necessarily match District standards.

5.2.4.1 Grated Catch Basins On-Grade

The capture efficiency of grated inlets on-grade depends on the width and length of the grate and the velocity of the flow approaching the grate. When the approaching flow velocity is slow and the flow width does not exceed the grate width, the grate inlet might be able to intercept all the approaching flow. In cases where the width of the approaching flow exceeds the grate width, very little of the approaching flow that exceeds the grate width will be intercepted by the inlet. When the velocity of the approaching flow is too high, the flow will "splash over" the grate. Both these phenomena contribute to bypass flow of grate inlets, which is comparable to the bypass flow discussed in relation to curb opening inlets on-grade.

The ratio of frontal flow to gutter flow, E_o for a straight cross slope is:

$$E_o = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \tag{5.16}$$

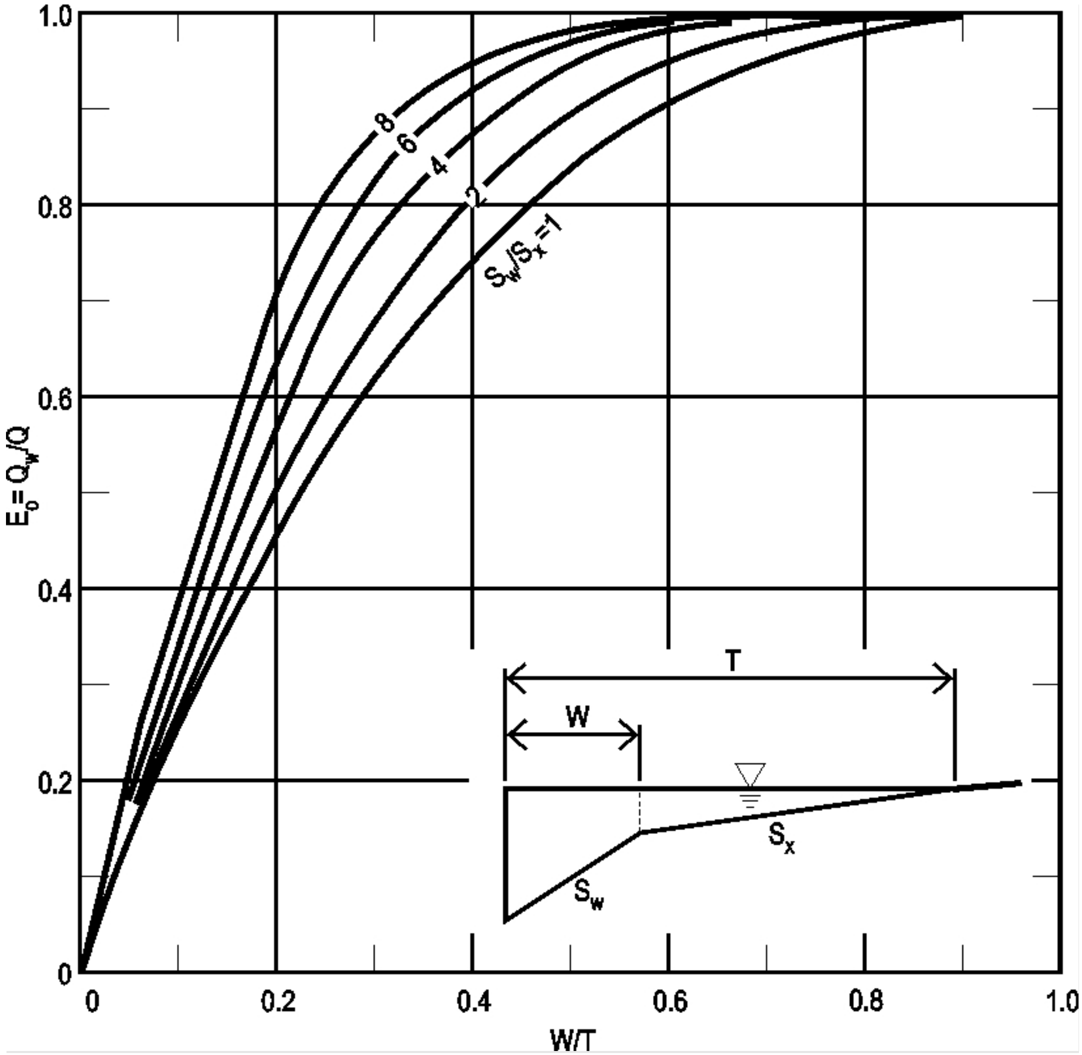
where:

E_o = ratio of frontal flow to gutter flow for a straight cross slope
Q_W = flowrate in width (W), (cfs)
Q = total flow (cfs)
W = width of grate (ft). The Federal Highway Administration's *Urban Drainage Design Manual* (HEC-22) provides guidance for grate types and configurations
T = spread of flow on the pavement (ft)

Figure 5.10 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections.

Figure 5.10: Ratio of Frontal Flow to Total Gutter Flow

(USDOT, FHWA, 1984, HEC-12, CHART 4)



The ratio of side flow, (Q_S) to total gutter flow (Q) is:

$$\frac{Q_S}{Q} = 1 - \frac{Q_W}{Q} = 1 - E_o \tag{5.17}$$

where:

Q_S = flowrate outside of width (W), (cfs)

Q_W = flowrate in width of grate or gutter (W), (cfs)

E_o = ratio of frontal flow to gutter flow for a straight cross slope

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed:

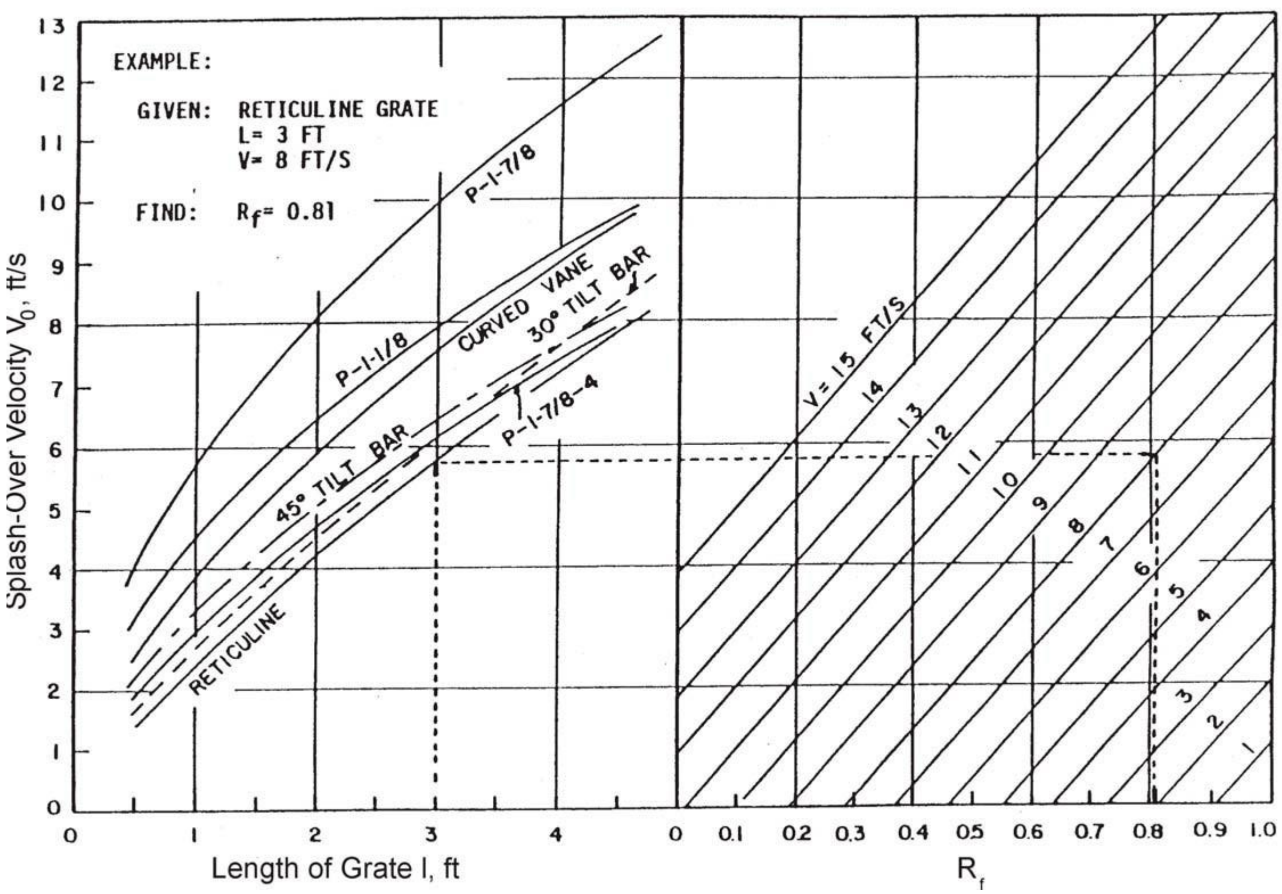
$$R_f = 1 - 0.09(V - V_o) \tag{5.18}$$

where:

R_f = ratio of frontal flow intercepted to total frontal flow
V = velocity of flow in the gutter (ft/sec)
V_O = gutter velocity where splash over first occurs (ft/sec)

This ratio is equivalent to frontal flow interception efficiency. Figure 5.11 provides a solution of Equation (5.18) which considers grate length, bar configuration, and velocity at which splashover occurs. The velocity when using Figure 5.11 is flow in the street half-width divided by the area of flow.

Figure 5.11: Grate Inlet Frontal Flow Interception Efficiency
(USDOT, FHWA, 1984, HEC-12, Chart 7)



The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed:

$$R_s = \frac{1}{1+\frac{0.15V^{1.8}}{S_x L^{2.3}}} \quad (5.19)$$

where:

R_S = ratio of side flow intercepted to total side flow
S_X = pavement cross slope (ft/ft)
L = length of grate (ft)
V = velocity of flow in the gutter (ft/sec)

Although a grate may intercept nearly all side flow where the velocity is low and the spread slightly exceeds the grate width, errors due to this deficiency is very small. Additionally, where velocities are high, side flow interception can be neglected entirely without significant error.

The efficiency, E, of a grate is:

$$E = R_f E_o + R_s(1 - E_o) \tag{5.20}$$

The first term on the right side of Equation (5.20) is the ratio of intercepted frontal flow to total gutter flow (R_f), and the second term is the ratio of intercepted side flow to total side flow (E_o). The second term is insignificant with high velocities and short grates.

The interception capacity (Q_i) of a grated catch basin on-grade is equal to the efficiency of the grate multiplied by the total street half-width flow:

$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)] \tag{5.21}$$

5.2.4.2 Grated Catch Basins in a Sump

The efficiency of catch basins in passing debris is critical in sump locations. Total or partial clogging of catch basins in these locations can result in hazardous ponding conditions. Grated catch basins alone are not recommended for use in sump locations because of the tendency of the grates to become clogged. Combination catch basins or curb-opening catch basins (no grate) are recommended for use in these locations.

A grated catch basin in a sump location may operate as a weir or orifice dependent on the bar configuration, size of the grate, and flow depth. Larger grates and grates with more open area, that is, with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than smaller grates or grates with less open area. **The designer shall calculate the capacity of the inlet under both weir flow and orifice flow conditions, then adopt a design capacity equal to the smaller of the two results.**

Inlets located in sump locations must be able to intercept the peak discharge from a 100-year design event. Inlets within sump locations require a secondary (emergency) outlet that is clear to the sky (i.e., a swale or ditch rather than a second catch basin) unless approved otherwise by the local agency. The depth of water in the emergency outlet must be at least one foot below adjacent finished floor elevations in cases where the flow path is between dwelling units. The emergency outlet must also be placed within a drainage easement to ensure conveyance is maintained in perpetuity. This emergency outlet system must direct overflows to either a downstream street with adequate capacity, a natural conveyance system, or an alternative system acceptable to the local agency. The point of discharge must also be analyzed to prevent downstream impacts.

The capacity of grated catch basins operating as weirs is:

$$Q_i = C_w P_e d^{1.5} \tag{5.22}$$

where:

Q_i = intercepted flow capacity of grated catch basin operating as a weir (cfs)
C_W = Weir coefficient = 3.0 (U.S. Traditional Units)
Pe = effective perimeter of the grate

d = depth of flow at curb (ft)

To account for the effects of clogging of a grated inlet operating as a weir, a clogging factor of fifty percent (C_L = 0.50) shall be applied to the actual (unclogged) perimeter of the grate (P):

$$P_e = (1 - C_L)P \quad (5.23)$$

where:

P_e = effective perimeter of the grate
C_L = clogging factor (C_L = 0.50)
P = actual grate perimeter subtracting out length occupied by bars and disregarding side against curb (ft)

The capacity of a grate catch basin operating as an orifice is:

$$Q_i = C_o A_e (2gd)^{0.5} \quad (5.24)$$

$$A_e = (1 - C_A)A \quad (5.25)$$

where:

Q_i = intercepted flow capacity of grated catch basin operating as an orifice (cfs)
C_O = Orifice coefficient = 0.67 for U.S. Traditional Units
A_e = effective (clogged) area of the grate (sq ft)
d = depth of flow at curb (ft)
g = acceleration due to gravity 32.2 ft/sec ^{2}
C_A = area clogging factor (C_A = 0.50)
A = actual opening area of the grate inlet (i.e., the total area less the area of bars or vanes). Orifice coefficient = 0.67 for U.S. Traditional Units

5.2.4.3 Clogging Factors

The engineer shall select an appropriate clogging factor for all catch basin inlets. At minimum, a clogging factor of 50% (50% reduction of capacity) shall be applied to all combination (curb opening + grate) inlets located in a sump or sag condition.

With regard to the selection of a clogging factor for all inlet types, a number of considerations must be contemplated. Attention must be given to the nature of the tributary area for sources of potential clogging, trash, leaves, etc., longitudinal street slopes, and corresponding flow velocity in determining how likely an inlet is to be clogged. In addition, the consequences of the catch basin's failure should be assessed when selecting a clogging factor to be applied, particularly in the flooding path that would result if the catch basin were to fail.

5.2.5 Combination Catch Basins

The Standard Riverside County combined curb inlet/grated catch basin is per District Standard Drawing CB102. Design Engineers should confirm with the local jurisdiction for the applicable catch basin standard drawing.

For all catch basin inlets, hydraulic analyses may be calculated by using: 1) accepted software, or 2) follow HEC-22 criteria with hand calculations. See examples using both hand calculations and software in Section 5.3.

5.2.5.1 Combination Inlets On-Grade

A combination catch basin on-grade is typically used when street grades exceed 5%. To reduce clogging of the grate, the inlet is configured with the majority of the curb opening located upstream of the grate and the remainder of the curb opening located adjacent to the grate. The procedure used to calculate capacity in this case is to first calculate the capacity of the curb opening inlet using the opening length upstream of the grate. Next, determine the depth, width, and velocity of the flow that would bypass the curb opening, and using that information, calculate the capture of the grated portion of the inlet using the procedures previously described for on-grade grated inlets.

5.2.5.2 Combination Inlets in a Sump

Combination catch basins consisting of a grate and a curb opening are considered advisable for use in sumps where hazardous ponding can occur. The interception capacity of the combination catch basin is essentially equal to that of a grate alone in weir flow unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

Equation (5.22) can be used for weir flow in combination catch basins in sump locations. Assuming complete clogging of the grate, Equation (5.11), Equation (5.13), and Equation (5.14), for curb-opening catch basins are applicable.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the catch basin is computed by adding Equation (5.24) and Equation (5.14), resulting in Equation (5.26):

$$Q_i = 0.67\,A_g(2gd)^{0.5} + 0.67hL(2gd_o)^{0.5} \tag{5.26}$$

where:

Q_i	=	Amount of street flow intercepted by inlet (cfs)
A_g	=	clear opening area of the grate (sq ft)
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
d	=	depth of flow at curb (ft)
h	=	height of curb opening portion of catch basin, curb opening orifice or orifice throat (ft)
L	=	length of curb opening (ft)
d_O	=	effective depth at the center of the curb opening orifice (ft)

5.2.6 Determination of Catch Basin Vault Depth

For preliminary analysis, the minimum catch basin vault depth, V, shall be determined as follows:

$$\mathrm{V} = \mathrm{C.F.} + 0.5 + 1.2\frac{v^2}{2g} + \frac{d}{\cos S} \tag{5.27}$$

where:

V	=	depth of the catch basin vault, or "V" depth, measured in feet from the invert of the connector pipe to the top of the curb (ft)
C.F.	=	vertical dimension of the curb face at the catch basin opening (ft)
v	=	average velocity of flow in the connector pipe, assuming a full pipe section (ft/sec)
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
d	=	diameter of connector pipe (ft)

S = slope of connector pipe (ft/ft)

The term 1.2 $v^2/2g$ includes an entrance loss of 0.2 of the velocity head.

A minimum of 0.5 feet of freeboard is required in the catch basin vault from the connector pipe 100-year water surface to the gutter flow line.

Assuming a curb face at the catch basin opening of 9 inches (6-inch curb face + 3-inch local depression), and Cos S = 1, Equation (5.27) may be simplified to the following:

$$V = 1.25 + 1.2\frac{v^2}{2g} + d \tag{5.28}$$

For detailed design of a catch basin vault depth, additional factors need to be considered. In many cases, the connector pipe may not be completely full or may have a higher HGL so the method presented herein is not applicable to all cases. For final design, the calculated HGL from varied flow or pressure flow calculations (typically WSPG) should be used with the freeboard and minimum pipe cover required over the connector pipe.

5.3 APPLICATION

This Section offers design examples for street drainage inlets. Equations presented in this chapter shall be used for design purposes.

5.3.1 Design Procedures

For the "on-grade" condition, the capacity of the inlet is dependent upon many factors including street and gutter longitudinal and cross slope, depth of flow, height and length of the opening, and the depth of gutter depression at the inlet. Guidelines regarding bypass flow are located in Section 5.2.3. Design procedures for street drainage on-grade are as follows:

1. Determine the Q10 and Q100 flowrate capacity of the street based on a given longitudinal street slope and cross section per Section 5.2.2. Confirm whether the flow is contained with half-street or spills over the crown.
2. Determine the tributary flow rate to the street per Chapter 3 .
3. Compare the street capacity flowrate determined in Step 1 to the tributary flowrate calculated in Step 2. If the flow rate from Step 2 exceeds the capacity from Step 1 (Q>capacity), pick another location further upstream. Continue the iterative process until the drainage area flow rate is consistent with the street capacity flow rate. Then initiate a catch basin or series of catch basins.
4. Determine if there are conflicts with the placement of a catch basin at this location. Conflicts include, but are not limited to, side streets, driveways, utilities that would be costly to relocate, etc. Should there be conflicts, move the catch basin location upstream.
5. Size a catch basin to intercept the calculated flow. Determine the efficiency of the catch basin and determine the flow rate, if any, that will bypass the catch basin.
6. Choose a location downstream in which the drainage area contributing to the location will generate a flow rate that when added to the bypass flow rate of the upstream catch basin

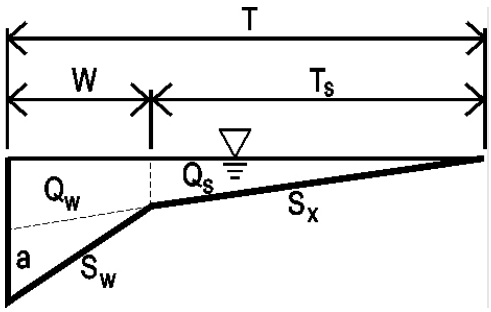
determined in Step 4 is equal to the flow rate that would generate a spread that is equal to the allowable spread.

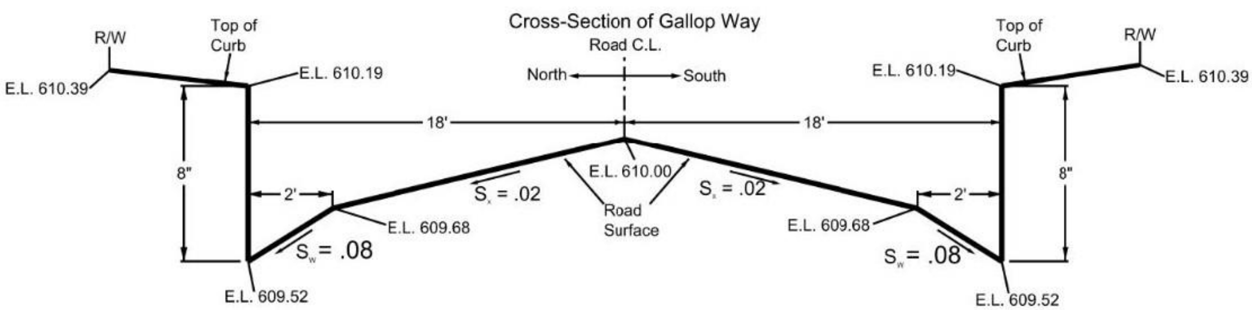
7. Continue steps 3 through 5 to termination of the project. Design examples for these procedures are shown in Section 5.3.2.

5.3.2 Design Example 1 – Street Flow

Example 1 (Hand Calculations)

From a 100-year hydrology study of the area it is known that 20 cfs is traveling down local street Gallop Way. If Gallop Way is an existing road, the capacity should be determined using actual surveyed cross sections of the road instead of the standard detail. Determine the total discharge (Q) traveling on the North Side of the street before spilling over the crown to the south side of the street.





Given:

Allowable spread	T	=	18 ft
Cross Slope	S_X	=	.02 ft/ft
Gutter depression (hike – Sx projection)	a_G’	=	1.5"-(2’x.02)=1.46"= 0.12’
Longitudinal Slope	S	=	0.013 ft/ft
Gutter Width	W	=	2.0 ft
Manning’s roughness value	n	=	0.015

Step 1:

Determine the flow spread (T_s) for the pavement and gutter section. The maximum allowable spread for the north side of Gallop Street is 18' before the crown will be overtopped.

$$T = W + T_s$$

$$T_s = T - W = 18\,ft - 2.0 = 16.0ft$$

Step 2:

Determine the discharge (Q_s) in the pavement section using Equation (5.4).

$$Q_s = \left(\frac{0.56}{n}\right) S_x^{1.67} S^{0.5} T_s^{2.67} \tag{5.4}$$

$$Q_s = \frac{0.56}{0.015} \times 0.02^{1.67} \times 0.013^{0.5} \times 16.0^{2.67} = 10.16\,cfs$$

Step 3:

Determine the total discharge (Q) using Equations (5.5) and (5.6).

$$E_o = 1\Bigg/\left(1 + \frac{S_W/S_x}{\left[1+\frac{S_W/S_x}{\frac{T}{W}-1}\right]^{2.67} - 1}\right) \tag{5.5}$$

To solve Equation (5.4), determine S_w, S_w/S_x and T/W.

$$S_w = \frac{a'_G}{W} + S_x = \frac{0.12\,ft}{2.0\,ft} + 0.02\,\frac{ft}{ft} = 0.08\,\frac{ft}{ft}$$

$$\frac{S_W}{S_x} = \frac{0.08}{0.02} = 4.0$$

$$\frac{T}{W} = \frac{18}{2.0} = 9.0$$

By substitution:

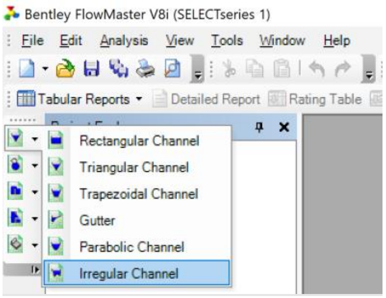
$$E_o = 1\Bigg/\left\{1 + \frac{4.0}{\left[1+\frac{4.0}{9.0-1}\right]^{2.67} - 1}\right\} = 0.328 \tag{5.5}$$

Use Equation (4.6) to solve for the total discharge using Q_s and E_o.

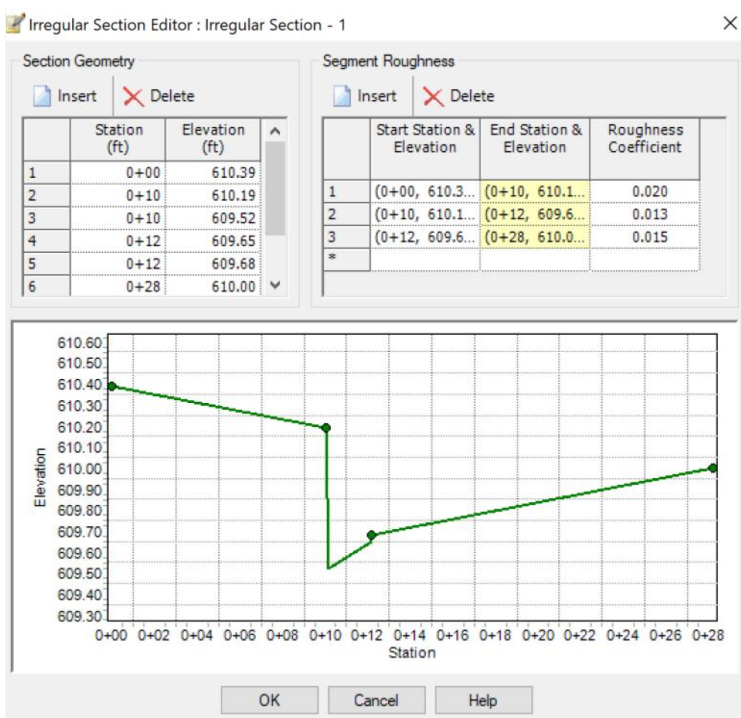
$$Q = \frac{Q_s}{(1-E_o)} = \frac{10.16}{1.0-0.328} = 15.12\,cfs \tag{5.6}$$

Example 1 (using FlowMaster software)

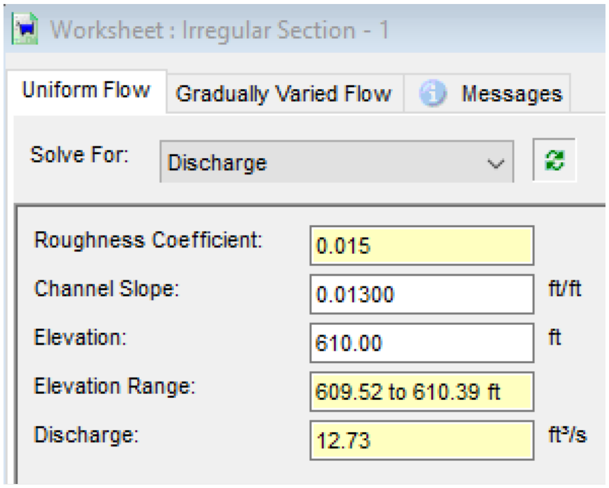
Use the 'Irregular Channel' application in FlowMaster to determine the capacity on the north side of the street.



Select the Edit Section button in the Irregular Section dialogue window and enter the geometry that represents the cross-section for the north side of Gallop Way based on proposed street section or on topographic (DTM) data for existing streets.



Solve for the capacity of the cross-section. In this example, the elevation (in dialogue box below) was selected assuming flow depth does not exceed the street crown. From the dialogue window, the capacity of the north side of Gallop Way is 12.73 cfs, say 13 cfs.



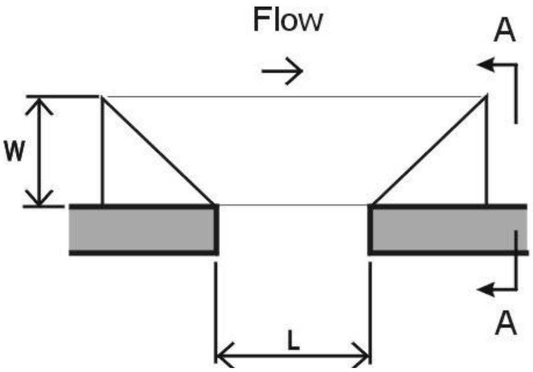
The design engineer must consider which side of the street will reach capacity first. For this example, it was assumed that the north side of Gallop Way would fill up first, meaning that 12.73 cfs of the tributary 20 cfs will flow on the north side, and the remaining 7.27 cfs will spill over the crown to the south side. Since the street cross section at this location was verified to be symmetrical, the south side of the street would flow at a depth below the crown. If the total flow was greater than the capacity below the crown for both sides of the street, the entire street section will need to be used to determine capacity to top of curb or right of way limits.

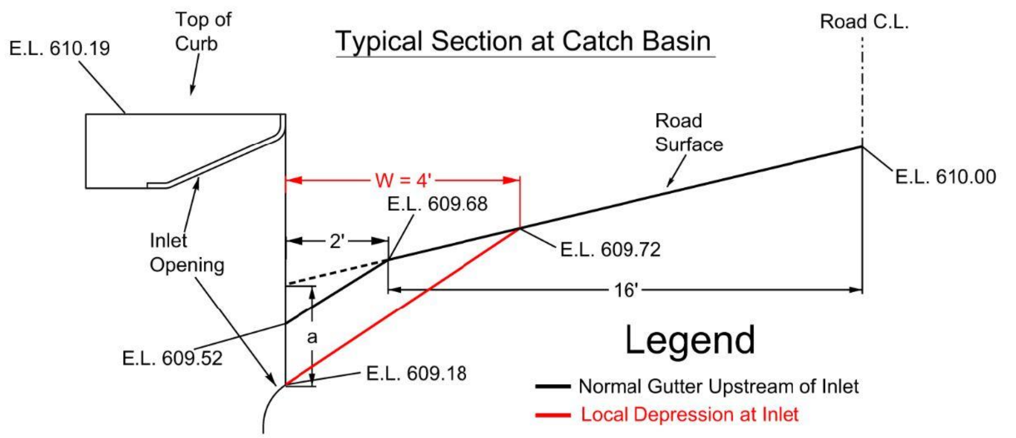
5.3.3 Design Example 2 – On-Grade Curb Inlet

Determine the length of a curb-opening inlet on-grade for the interception of the street and gutter flow on the north side of Gallop Way determined in Example 1 and whether it fits in the available space.

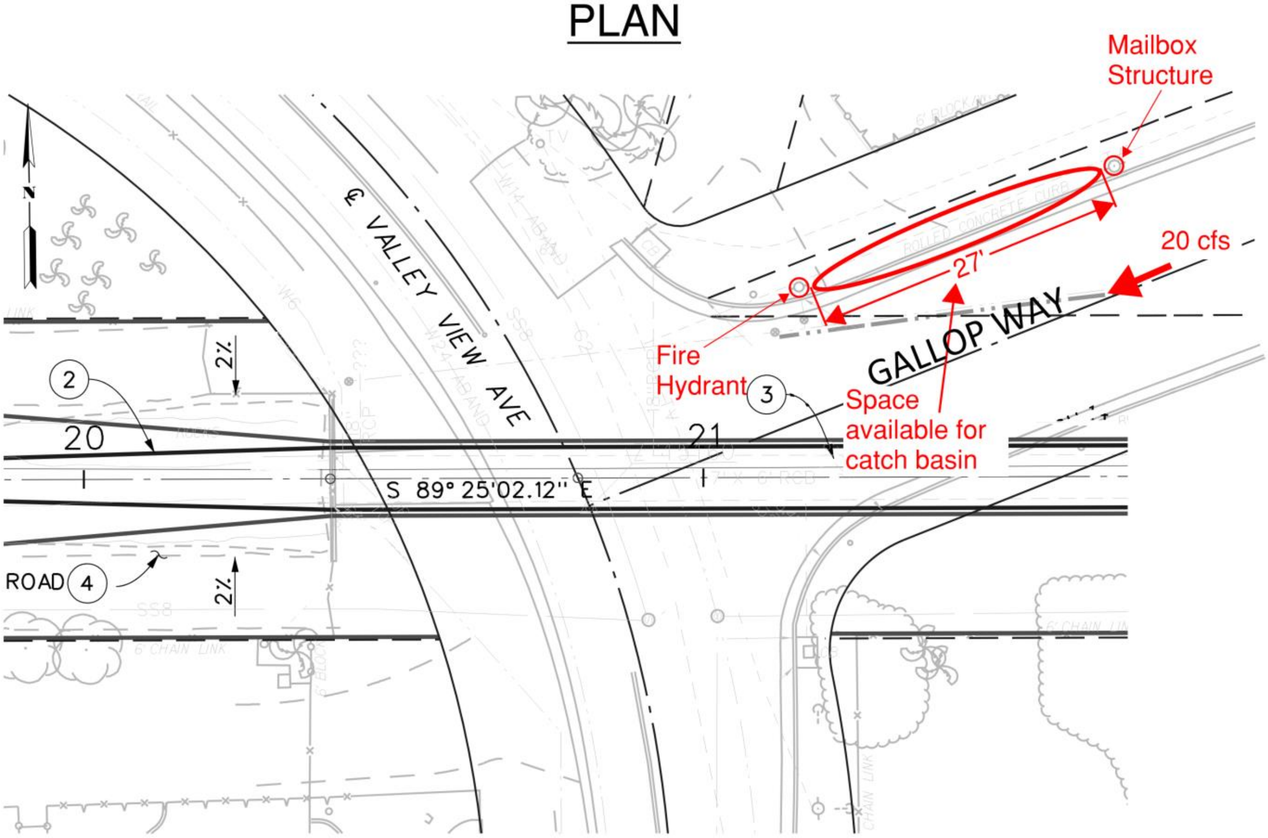
Given:

Design flow on north side of Gallop Way	Q	=	13 cfs
Cross Slope	S_X	=	.02 ft/ft
Gutter depression (hike – Sx projection)	a'_G	=	1.5"-(2'x.02)=1.46"= 0.12'
Longitudinal Slope (from problem statement)	S	=	0.013 ft/ft
Gutter Width (upstream of Catch Basin)	W	=	2.0 ft
Manning's roughness value	n	=	0.015
Local depression at inlet	a'_D	=	4" = 0.33' (Example based on previous standard of 4") (Note: TLMA Std 311/312 = 3")
Local depression Gutter Width	W	=	4.0 ft (RCFC Std LD201)
Combined depression (a'_G + a'_D)	a	=	0.45 ft
Allowable spread	T	=	18 ft
Gutter Cross Slope (609.68-609.52)/2	S_W	=	0.08 ft/ft (from typical section)


Flow
A
W
L
A


Typical Section at Catch Basin
Top of Curb
E.L. 610.19
Road C.L.
Road Surface
E.L. 610.00
W = 4'
E.L. 609.68
2'
E.L. 609.72
Inlet Opening
16'
a
E.L. 609.52
E.L. 609.18
Legend
Normal Gutter Upstream of Inlet
Local Depression at Inlet

PLAN


Mailbox Structure
20 cfs
27'
℄ VALLEY VIEW AVE
GALLOP WAY
Fire Hydrant
Space available for catch basin
2
3
4
20
21
ROAD
2%
2%
S 89° 25'02.12'' E

Example 2 (Hand Calculations)

Step 1:

Determine the equivalent cross-slope using Equation (5.9). Note, use gutter depression at inlet.

$$S_e = S_x + S'_w E_o \qquad (5.9)$$

$$S'_w = \frac{a}{W} = \frac{0.45\,ft}{4.0\,ft} = 0.11\frac{ft}{ft}$$

$$E_o = 1/\left\{1 + \frac{0.1325/0.02}{\left[1+\frac{6.625}{4.5-1}\right]^{2.67} - 1}\right\} = 0.71 \qquad (5.5)$$

$$S_e = 0.02\frac{ft}{ft} + \left(0.11\frac{ft}{ft} \times 0.71\right) = 0.10\ \frac{ft}{ft}$$

Step 2:

Using Equation (5.8) solve for length.

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \qquad (5.8)$$

$$L_t = 0.6 \times 13^{0.42} \times 0.013^{0.3} \times \left(\frac{1}{0.015 \times 0.10}\right)^{0.6} = 23.7\ ft$$

The District's standard catch basin sizes are 7', 14', 21', and 28'. Therefore, the recommended catch basin length to intercept flows on the north side of Gallop Way is 21 feet.

Step 3:

The recommended curb opening is 21 feet; therefore, determine the catch basin efficiency (E), the flow intercepted (Q_i) and the bypass flow.

Use Equation (5.10) to determine the efficiency of the catch basin provided.

$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \tag{5.10}$$

$$E = 1 - \left[1 - \frac{21}{22.4}\right]^{1.8} = 0.99$$

$$Q_i = Q \times E$$

$$Q_i = 13\ cfs \times 0.99 = 12.9\ cfs$$

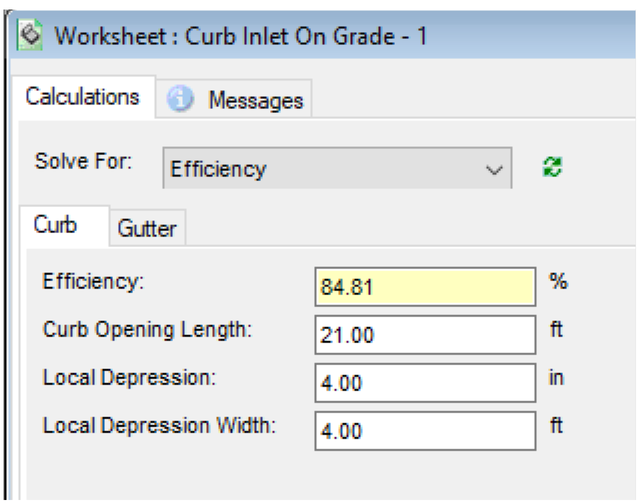
$$Q_{bypass} = Q - Q_i$$

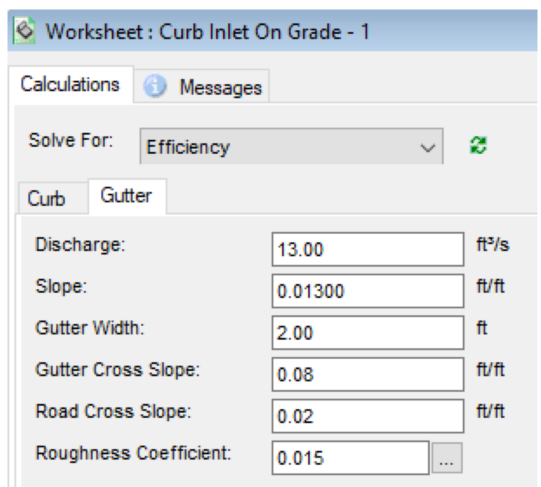
= 13 cfs – 12.9 cfs

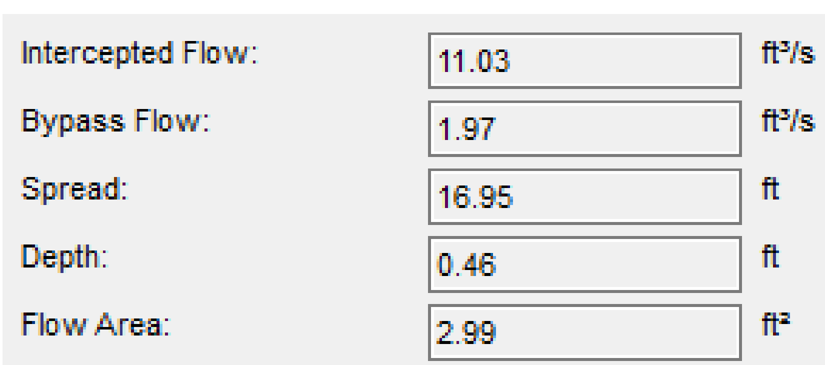
= 0.1 cfs

Example 2 (Using FlowMaster Software)

Based on the following inputs into FlowMaster, the calculated flow intercepted is 11.0 cfs with 2 cfs bypassed.







Example 2 (using CivilD software)

It should be noted the CivilD is not a pure implementation of HEC-22 for determining street flow characteristics (spread, depth, velocity, etc.); CivilD implements a pure form of Manning's Equation. Whereas HEC-22 proposes the use of a modified version of Manning's Equation specifically integrated for use in triangular channels. The results obtained from CivilD will not be completely identical to results obtained by HEC-22, however, in most cases the results are similar. Results derived from either method will be accepted. It should also be noted that CivilD assumes sag conditions for longitudinal street slopes less than 0.5%; the engineer should determine whether this assumption is acceptable.

Manning's Equation

$$Q=\left(\frac{1.49}{n}\right)AR^{0.67}S^{0.5} \tag{5.1}$$

Modified Manning's Equation

$$Q_t = \left(\frac{0.56}{n}\right) S_x^{1.67}S^{0.5}T^{2.67} \tag{5.2}$$

Launch CivilD Hydrology/Hydraulics Programs, select Option 5 General Hydraulics (Street, Inlet). Create or modify study file using Option 4 (Street flow + Street Inlet Analysis).

CivilD Input	Variable	Description	Value for this example
Flow Rate			
Enter maximum flow rate at headworks	Q	Design flow rate tributary to catch basin	13cfs
Longitudinal Slope			
Enter <system headworks> invert elevation	-	Enter flowline elevation upstream of catch basin	609.72' (select value to get S_L = 0.013)
Enter <system outlet> invert elevation	-	Enter flowline elevation downstream of catch basin	609.33' (select value to get S_L = 0.013)
Enter length or distance between above points	-	Enter length between the two flowline elevations	30' (select value to get S_L = 0.013)
Street Geometry Data Entry			
Enter curb height above	H_c	Height of curb upstream of	8"

CivilD Input	Variable	Description	Value for this example
gutter flowline		catch basin	
Enter the distance from curb to property line	D_{back}	Distance from curb face to R.W.	10’
Enter width of half street from curb face to crown	D_{half}	Centerline to curb face	18’
Enter distance from street crown to grade break	D_{break}	If not applicable enter distance ≤ D_{half} - W_D	14’ (since no grade break present = street half-width minus local depression width)
Enter the gutter width	W_G	Width of gutter upstream of catch basin	2’
Enter the gutter hike	a’G	See Figure 5.8	1.5"
Enter the slope <vert/horiz> from curb to property line	S_{back}	Slope from curb face to R.W.	.02
Enter slope from grade break to crown	S_X	Street cross slope	0.02
Enter slope from curb gutter to grade break	S_X or S_{X1}	If not grade break, enter same value as street cross slope	0.02
Select streetflow option desired	-	Follow program guidance	
Curb Inlet Data Entry			
Enter the inlet length	L	Either 7’, 14’, 21’ or 28’	21’
Enter the width of street depression (at catch basin)	W_D	Typically, 4’ per TLMA Std No. 311	4’
Enter the depth of street depression	a’D	Typically, 3" per TLMA Std No. 311, See Figure 5.8. Example uses 4" per	4"

CivilD Input	Variable	Description	Value for this example
		previous standard	
Enter the height of curb inlet	h	See Figure 5.8; and Table 5.2 for inclined throat	11"

Create an output file to view the results. The curb opening inlet length required to capture the total north side street flow is 26.77 ft. The 21' curb inlet will capture 12.17 cfs with 0.82 cfs bypassing the inlet.

Note: CivilD assumes In-Sump conditions if the longitudinal street slope is less than 0.5%.

```
Half street cross section data points through curb inlet:
             X-coordinate (Ft.)   Y-coordinate (Ft.)
             0.0000                 1.2000 right of way
            10.0000                 1.0000 top of curb
            10.0000                 0.0000 flow line
            14.0000                 0.4583 gutter/depression end
            14.0000                 0.4583 grade break
            28.0000                 0.7383 crown
 Length required for total flow interception = Lt
 Lt = .6 * Q^0.42 * Slope^.3 * (1/(n*Se)^.6 =     26.777(Ft.)
 where Manning's n = 0.0150 and Slope = street slope =  0.0130
 Se = Equivalent Street x-slope including depression =  0.0815
 Efficiency = 1 - (1-L/Lt)^1.8 =  0.9367

Remaining flow in street below inlets =      0.822(CFS)
Depth of flow =   0.202(Ft.)
Average velocity =   1.928(Ft/s)
Total flow rate in 1/2 street =   0.822(CFS)
Streetflow hydraulics:
Halfstreet flow width (curb to crown) =   5.845(Ft.)
Average flow velocity =   1.93(Ft/s)
```

5.3.4 Design Example 3 – On-Grade Grate

Determine the interception capacity of a single grated inlet on-grade for the flow rate determined in Example 1.

Given:

Design flow on north side of Gallop Way	Q	=	13 cfs
Flow in pavement section	Q_S	=	10.16 cfs (from Example 1)
Cross slope	S_X	=	0.02 ft/ft
Gutter depression	$a_G{}'$	=	0.12 ft
Cross slope of a depressed gutter ft/ft	S_W	=	0.08 ft/ft (from Example 1)
Longitudinal slope	S	=	0.013 ft/ft
Gutter Width	W	=	2.0 ft
Manning's Roughness Coefficient	n	=	0.015
Ratio of flow in the depressed section to total gutter flow	Eo	=	0.33 (from Example 1)

Allowable spread T = 18 ft

Table 5.3: Grate Parameters Chart

Grate Standard	RCFC	TLMA 24-18	TLMA 24-12	TLMA 24-9
Grate Width	2’	2’	2‘	2’
Grate Length	3.36’	3.45’	3.45’	3.45’
Grate Type	P-30mm (P-1-7/8")	P-30mm (P-1-7/8")	P-30mm (P-1-7/8")	P-50mm (P-1-7/8")

Example 3 (Hand Calculations)

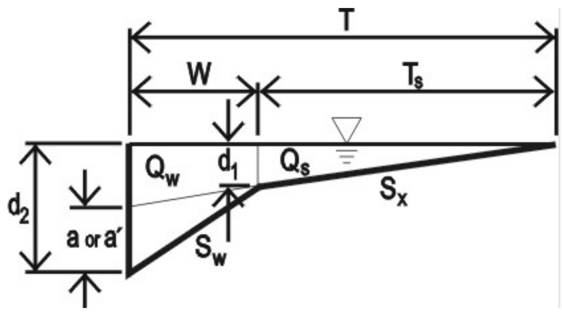
Step 1:

Determine flow rate Q_W in gutter width:

$$Q_W = Q - Q_s$$
$$= 13 - 10.2 = 2.8\ cfs$$

Step 2:

Determine velocity of flow in gutter width, W.



Determine d_1 and d_2 in figure above.

$$d_1 = (T - W)S_x = (18\ ft - 2ft) \times 0.02\frac{ft}{ft} = 0.32\ ft$$

$$d_2 = TS_x + a'_G = 18ft\ \times 0.02\frac{ft}{ft} + 0.12\ ft = 0.48\ ft$$

Determine flow area of Q_w.

$$\text{Flow area, } A_W = d_1 \times W + \left(\frac{(d_2 - d_1)\times W}{2}\right) \tag{5.29}$$

$$= (0.32\ ft\ \times 2\ ft) + \left(\frac{(0.48ft - 0.32\ ft)\times 2\ ft}{2}\right)$$

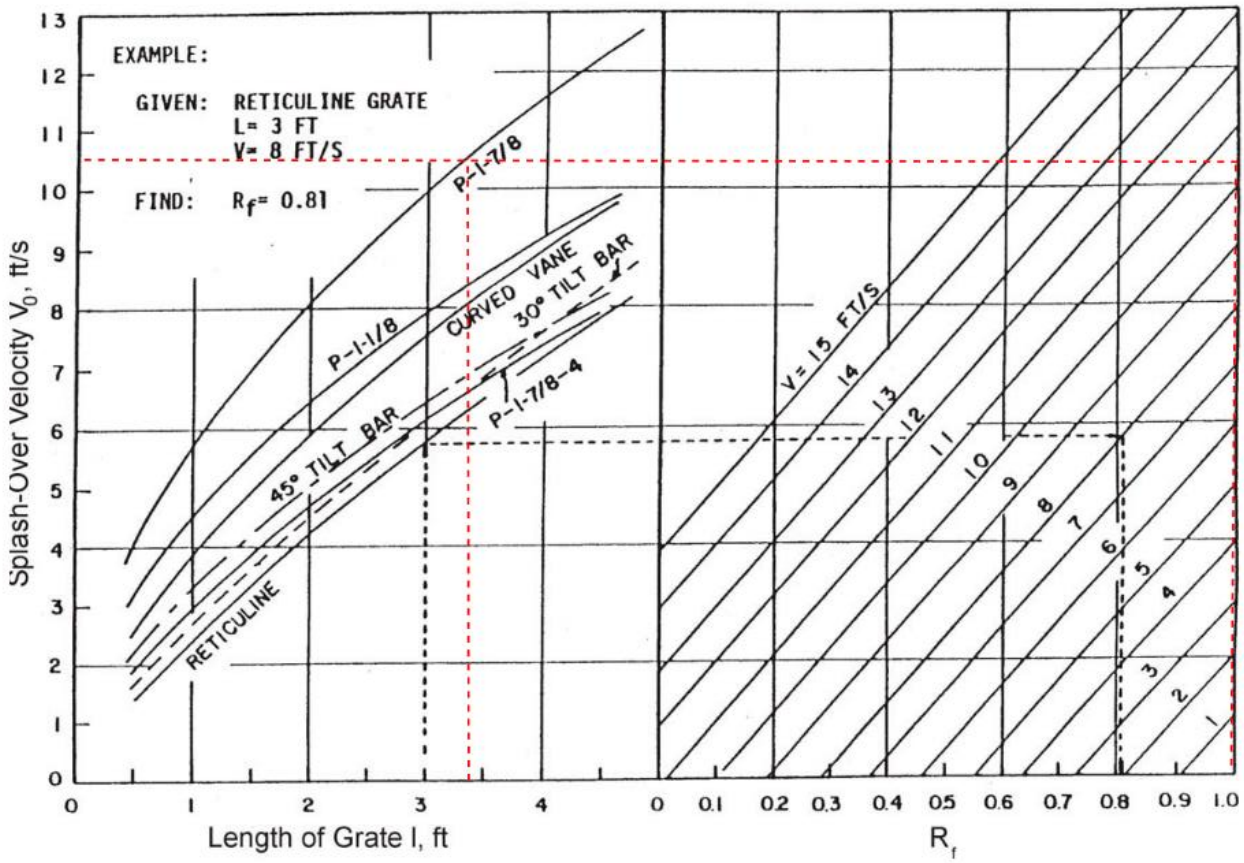
$= 0.8$ *sq ft*

Use $Q_W = VA_W$ to determine velocity.

$$\frac{Q_w}{A_w} = V = \frac{2.8\ cfs}{0.8\ sq\ ft} = 3.5\ fps$$

Step 3:

Determine splash over velocity (V_o) from Figure 5.11.



Length of grate = 3.36 feet, extend vertically from the length of grate value a line to the *P-1-7/8* curve, then extend a line horizontally to the splash-over velocity axis, read value.

$$V_o = 10.5\ fps$$

Step 4:

Using Equation (5.17) or Figure 5.11 determine the ratio of frontal flow intercepted to total frontal flow.

$$R_f = 1 - 0.09(V - V_o) = 1 - 0.09(3.5\ fps - 10.5\ fps) \qquad (5.18)$$

$= 1.6$ *say* 1.0 *or* 100%

With a clogging factor of 50%, the width of opening perpendicular to flow is 0.5 times the actual width of the grate. Therefore, R_f actual is equal to R_f with clogging $R_f \times 0.5 = 0.5$.

To use Figure 5.11 to determine R_f, extend a line vertically from the length of grate value to the *P- 1-7/8* curve, then extend a line horizontally to the diagonal V line to the value determined in

Step 2 and then vertically down to the R_f axis, read value. Maximum R_f value is equal to 1.

Step 5:

Using Equation (5.18) determine the ratio of side flow intercepted to total side flow, applying a 50% clogging factor to length of grate, L.

$$R_s = \frac{1}{1+\frac{0.15V^{1.8}}{S_x(L\times Clogging\ factor)^{2.3}}} = \frac{1}{1+\frac{0.15\,(3.5)^{1.8}}{(0.02)(3.36\times0.5)^{2.3}}} = 0.04 = \qquad (5.19)$$

Step 6:

Using Equation (5.19) determine the efficiency of the grate.

$$E = R_f E_o + R_s\,(1 - E_o) \qquad (5.20)$$

= 0.5 x 0.33 + 0.04 x (1- 0.33)

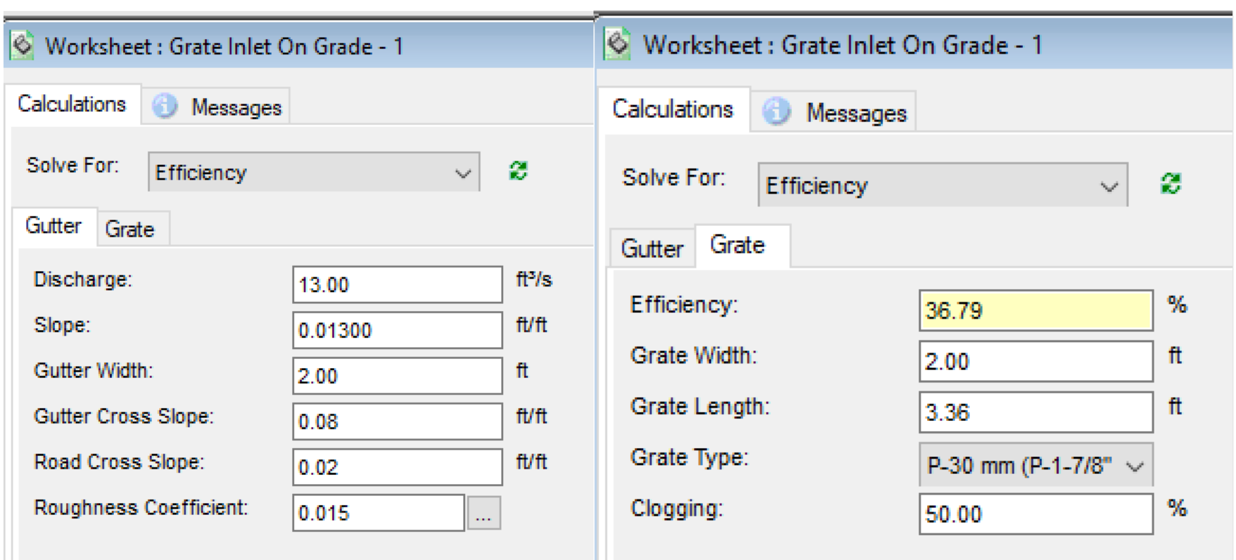
= 0.19

Step 7:

Determine flow rate Q_i intercepted

$$Q_i = (Q)(E) = 13 \times 0.19 = 2.5\,cfs \qquad (5.21)$$

Example 3 (Using FlowMaster Software)



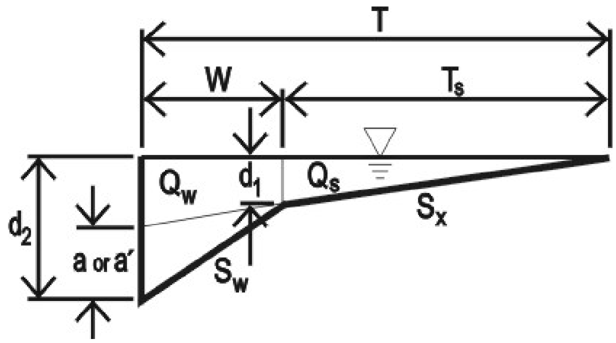
Intercepted Flow:	4.78	ft^3/s
Bypass Flow:	8.22	ft^3/s
Spread:	16.95	ft
Depth:	0.46	ft
Flow Area:	2.99	ft^2

5.3.5 Design Example 4 – On-Grade Combination Curb Inlet and Grate

Determine the capacity of a combination curb opening inlet for the flow rate determined in Example 1.

Given:

Design flow on north side of Gallop Way	Q	=	13 cfs
Cross slope	S_X	=	0.02 ft/ft
Gutter depression	a_G'	=	0.12 ft
Longitudinal slope	S	=	0.013 ft/ft
Gutter Width	W	=	2.0 ft
Manning's Roughness Coefficient	n	=	0.015
Total curb opening length		=	21 ft
Curb opening length upstream of grate		=	17.6 ft
Grate length (see Table 5.3)		=	3.36 ft
Gutter depression at inlet	a_D'	=	4 inches or 0.33 ft (Note: TLMA Std No. 311 uses 3" or 0.25 ft)
Local Depression Width		=	4 ft
Grate Type		=	P-30mm (P-1-7/8")
Grate Width (per Table 5.3)		=	2 ft
Grate Length (per Table 5.3)		=	3.36 ft
Curb Opening Length		=	21 ft
Grate clogging factor		=	50 percent



Example 4 (Hand Calculations)

Step 1:

Compute the interception capacity (Q_{ic}) of the curb opening upstream of the grate.

From Example 2: L_t = 22.4 ft

Use Equation (5.10) to determine efficiency of curb opening.

$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \quad (5.8)$$

$$E = 1 - \left(1 - \frac{17.64}{22.4}\right)^{1.8} = 0.93$$

$$Q_{ic} = Q \times E$$

$$Q_{ic} = 13\ cfs \times 0.93 = 12.1\ cfs, say\ 12\ cfs$$

Step 2:

Determine interception capacity (Q_{ig}) of the grate.

Flow to grate $Q_g = Q - Q_{ic} = 13\ cfs - 12\ cfs = 1.0\ cfs$

Step 2.1:

By assuming the flow spread T_s calculate the discharge Q_s in the paved section adjacent to grate using the procedure listed in Example 1 Step 2. This is an iterative process.

Assume: Ts = 1.2 *ft*

Q_S = 0.01 *cfs* (5.4)

Step 2.2:

Determine the total discharge following procedures listed in Example 1 Step 3. Note: Use gutter depression-value at inlet.

$$S_w = \frac{0.12+0.33}{4.0} + 0.02 = 0.133 \frac{ft}{ft}$$

$$\frac{S_w}{S_x} = \frac{0.133}{0.02} = 6.65$$

$$\frac{T}{W} = \frac{4.0+1.2}{4.0} = 1.3$$

$$E_o = 1/\left\{1 + \frac{6.65}{\left[1+\frac{6.65}{1.3-1}\right]^{2.67} - 1}\right\} = 0.99$$

$$Q_t = \frac{Q_s}{(1-E_o)} = \frac{0.01}{(1-0.99)} = 1\ cfs$$

Q_t from Step 2.2 equals Q_g from Step 2 therefore the assumption of T_s = 1.2 feet in Step 2.1 is correct. Should Q_t not equal Q_g, a different value for T_s would need to be assumed.

Step 3:

Determine flow rate (Q_w) in gutter width.

$$Q_w = Q - Q_s$$

$$= 1.0\ cfs - 0.01\ cfs = 0.99\ cfs$$

Step 4:

Determine velocity of flow in gutter width using procedures listed in Example 3, Step 2.

Note: $T = T_S + W = 1.2\ ft + 4\ ft = 5.2\ ft$

$$d_1 = 0.024\ ft$$

$$d_2 = 0.55\ ft$$

Flow area = 1.15 sq ft (4.27)

$$V = 0.9\ fps$$

Step 5:

Determine splash over velocity (V_o) from Figure 5.11.

$$V_o = 10.5\ fps$$

Step 6:

Determine the ratio (R_f) of frontal flow intercepted to total frontal flow for the grate. Use procedures listed in Example 3, Step 4.

R_f = 1.5 *if greater than 1, say 1*

With a combination curb opening and grate, a 50% clogging factor is applied to the grate, so R_f = 0.5.

Step 7:

Determine the ratio (R_s) of side flow intercepted to total side flow for the grate. Use procedures listed in Example 3, Step 5. 50% clogging factor applied.

$$R_s = 0.35 \qquad (5.19)$$

Step 8:

Using procedures listed in Example 3, Step 6 determine efficiency of the grate.

$$E = R_f E_o + R_s(1 - E_o)$$

$$E = (0.5)(0.99) + (0.35)(1 - 0.99) = 0.5$$

Step 9:

Determine the flow rate (Q_{ig}) intercepted by the grate.

$$Q_{ig} = Q \times E = 1cfs \times 0.5 = 0.5\ cfs$$

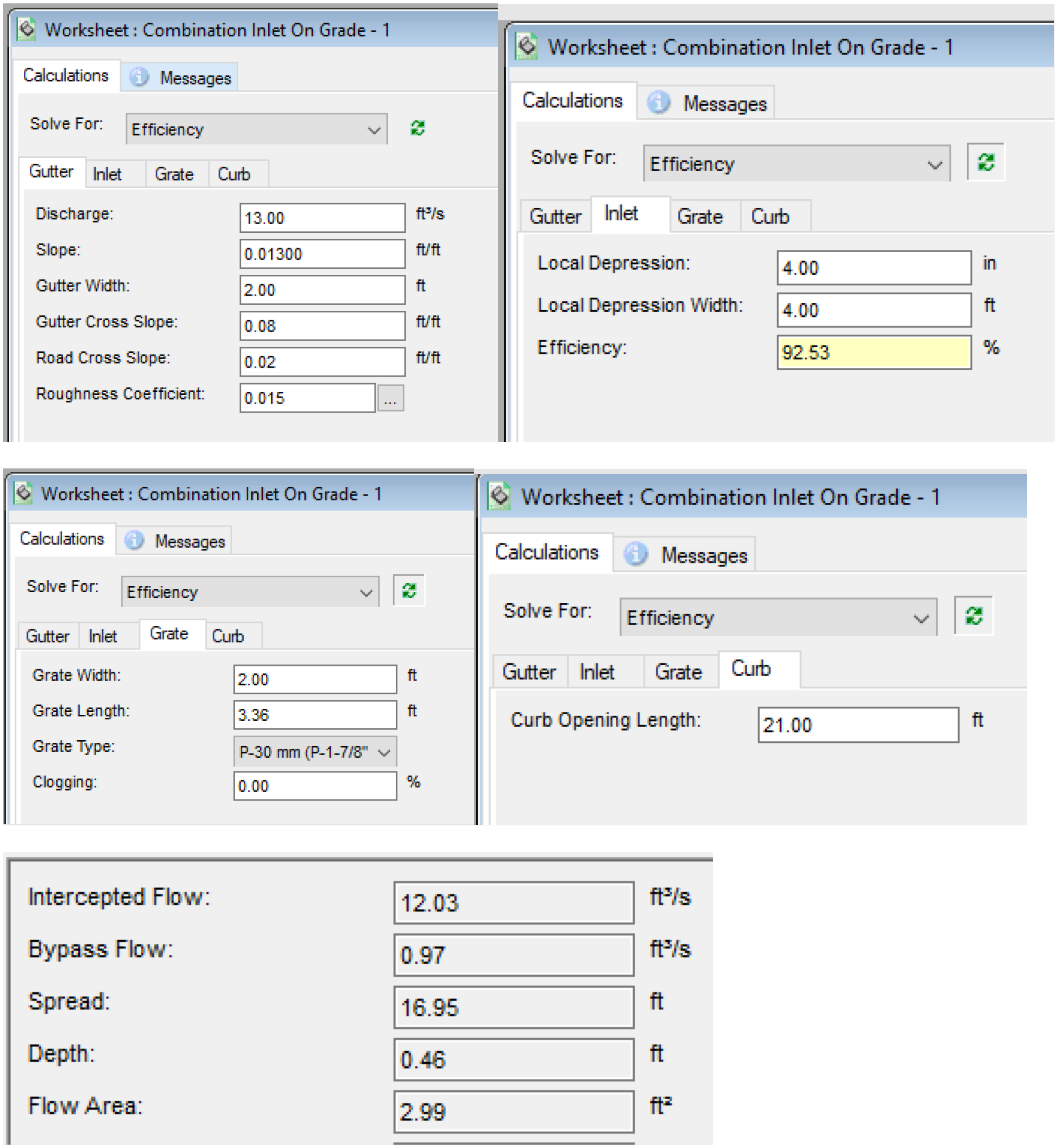
Step 10:

Determine the total flow (Q_i) intercepted by the combination catch basin.

$$Q_i = Q_{ic} + Q_{ig} = 12\ cfs + 0.5\ cfs = 12.5\ cfs$$

$$Q_{bypass} = Q - Q_i = 13\ cfs - 12.5\ cfs = 0.5\ cfs$$

Example 4 (FlowMaster Software)

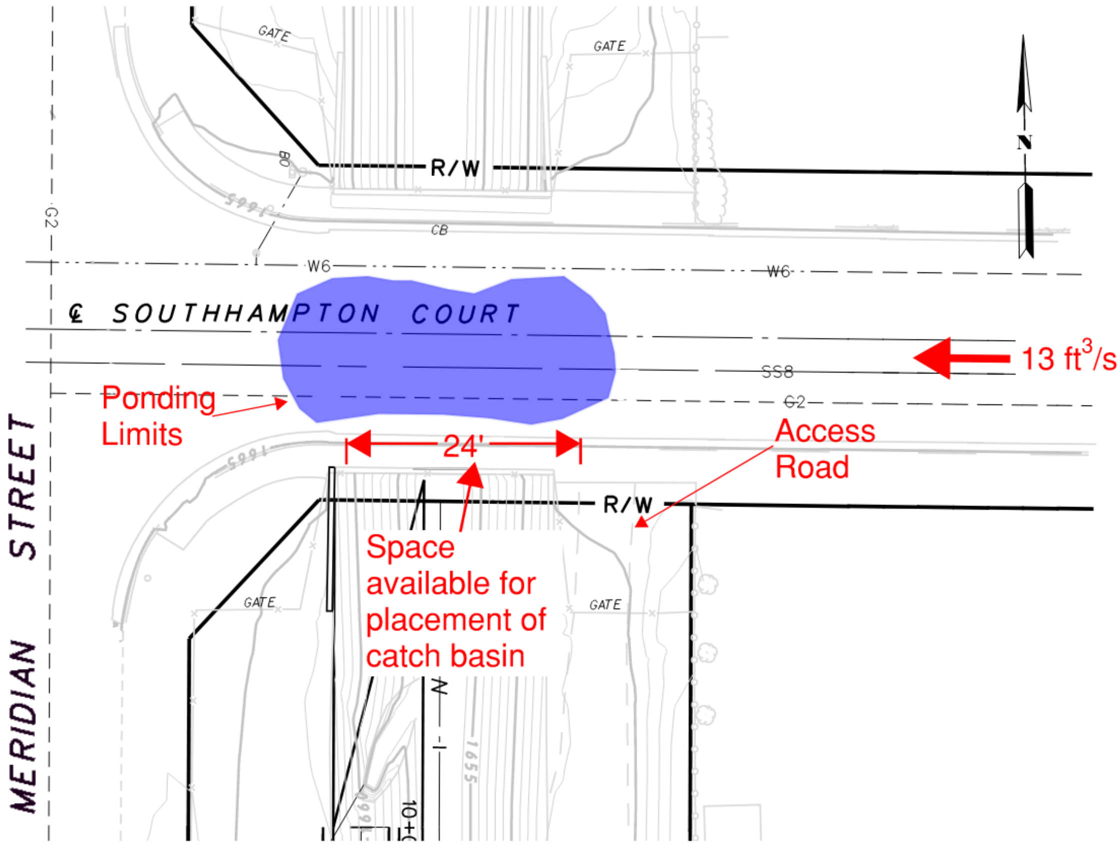


From Example 2, curb opening inlet intercepts 11.0 cfs whereas the combination inlet intercepts 12.0 cfs. Engineering judgement must be used to determine which inlet type is best warranted for use in this scenario.

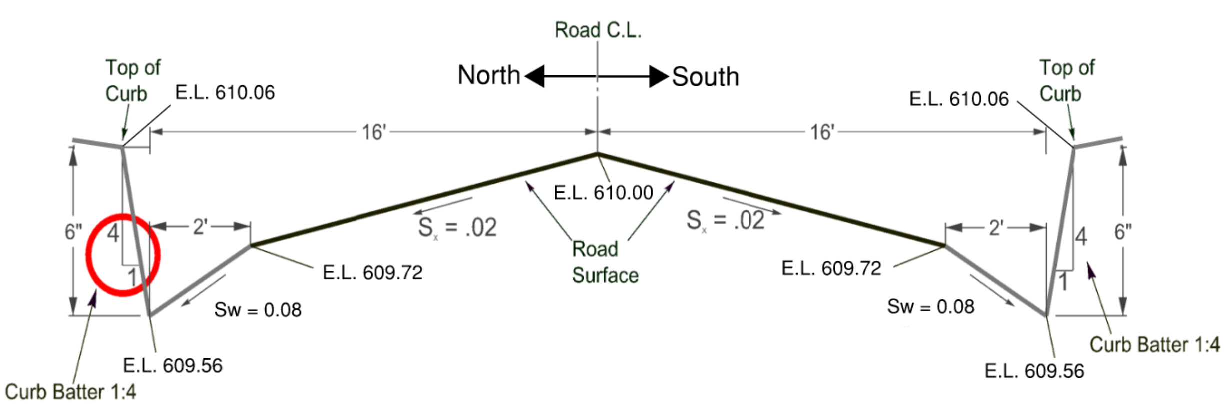
Adding additional grates to the combination inlet may reduce the flow intercepted from when there was only a single grate. This is because for combination inlets in the on-grade condition HEC-22 ignores the intereption capacity of the curb opening inlet length that is directly adjacent to the grates. So there is a trade-off that occurs when adding a grate in front of a curb opening inlet, in that the more grates that are added, the less the effective length of the curb opening becomes.

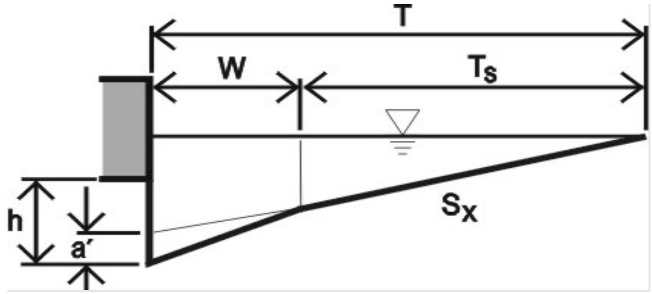
5.3.6 Design Example 5 – Sump Curb Inlet

From a 100-year hydrology study of the area it is known that 13 cfs is traveling down the south side of South Hampton Court. During recent storm events it has been observed that hazardous ponding is occurring near the intersection of Meridian Street and South Hampton Court. A catch basin is needed to alleviate the ponding conditions, determine the catch basin length required to eliminate the hazardous ponding conditions.



Cross-Section of South Hampton Court





Given:

Design flow on south side of South Hampton Ct	Q	=	13 cfs
Cross slope	S_X	=	0.02 ft/ft
Allowable spread	*T*	=	16 ft
Gutter depression at inlet	a	=	0.45 ft
Width of gutter at inlet	W	=	4.0 ft
Height of curb opening (see Figure 5.8, Table 5.2)	h	=	8.96 in = 0.75 ft
Weir coefficient	C_W	=	2.3

Ponding conditions are a project specific attribute that should be determined with deliberate consideration given toward the amount of flow contributing to the pond. Some useful tools in evaluating the extent of a pond may be found in the Hydrology and Hydraulics Application Add-In, in Inroads the 'Pass through Contours' tool. Applications in FlowMaster can also aid in determining the dimensions of a pond.

In cases like this problem where a pond forms in an irregular shape, spread becomes an irrelevant term for describing the ponding conditions as the spread varies throughout the pond. Sometimes it becomes more helpful to establish an acceptable depth for the ponding.

For this example problem, the acceptable depth will be set at the road crown. This depth is measured from the difference in elevations between the road crown and gutter depression. The acceptable ponding depth then equals 610' - 609.56' = 0.44 feet.

Example 5 (Hand Calculations)

Step 1:

Determine depth at inlet (d_i)

$$d_i = (S_x) \times (T)$$

$$d_i = (S_x) \times (T) = 0.02\frac{ft}{ft} \times 16\ feet = 0.32\ feet$$

Step 2:

Check that

$$d_i < h + \mathrm{a}$$

$$0.32\ ft < 0.75 + 0.45$$

$0.32\ ft\ < 1.2\ ft$

Step 3:

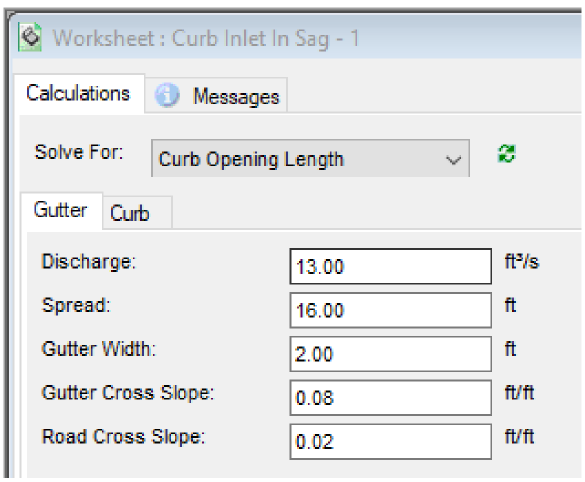
Weir condition since di < h+a, therefore, use Equation (5.11) to determine length. Use Equation (5.14) for orifice condition.

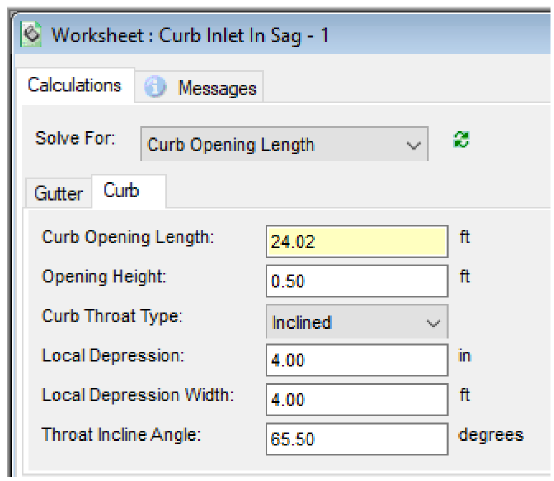
$$Q_i = C_w(L + 1.8W)d^{1.5} \tag{5.11}$$

or

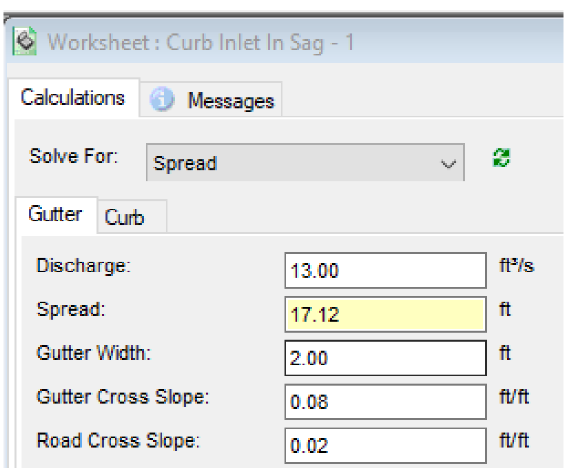
$$L = \frac{Q_i}{C_w d_i^{1.5}} - 1.8\ W = \frac{13\ cfs}{(2.3)(0.32^{1.5})} - (1.8)(4.0\ feet) = 24.0 feet$$

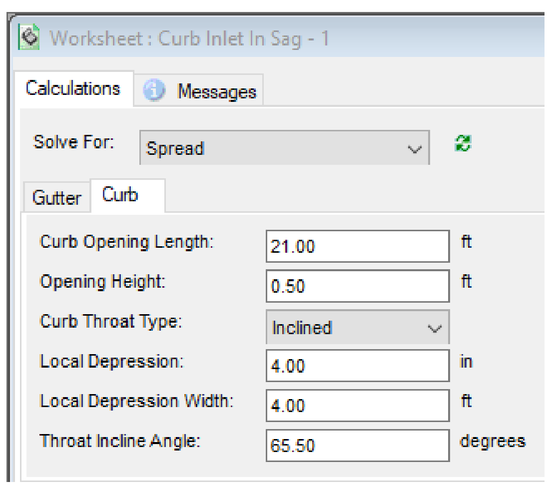
Example 5 (FlowMaster Software)





Due to the constraints on available space, the curb opening inlet length is limited to 21'.





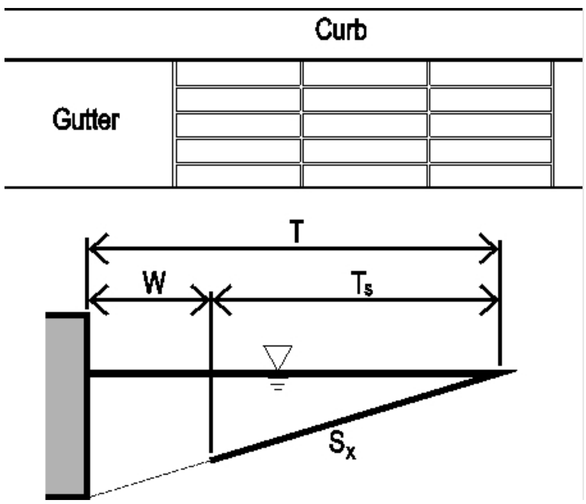
Depth:	0.46	ft
Gutter Depression:	0.12	ft
Total Depression:	0.45	ft

The depth of 0.46' is slightly greater than the previously determined accepted depth of 0.44' so the 21' curb opening inlet does not fully alleviate the harmful ponding conditions.

To increase the flow captured, a combination inlet (curb opening + grate) should be considered.

5.3.7 Design Example 6 – Sump Grate Inlet

Determine size of a grate inlet in a sump condition required based on information provided in Example 5.



Given:

Design flow on south side of South Hampton Ct	Q	=	13 cfs
Cross slope	S_X	=	0.02 ft/ft
Allowable spread	T	=	16 ft
Gutter depression at inlet	a	=	0.45 ft
Width of gutter at inlet (local depression)	W	=	4.0 ft
Weir coefficient	C_W	=	3.0
Orifice coefficient	Co	=	0.67
Grate type (see Table 5.3 RCFC)		=	P-30mm (P-1-7/8")
Grate dimensions (see Table 5.3 RCFC)		=	3.36 ft by 2 ft
Clogging factor (standard for all grated inlets)	C_L	=	50 percent
Gutter Width		=	2 ft

Example 6 (Hand Calculations)

Step 1:

Determine depth at inlet (d)

$$d = (S_x) \times (T) = 0.02\frac{ft}{ft} \times 16.0\,feet = 0.32$$

Step 2:

Substitute Equation (5.23) into Equation (5.22) to solve for P for weir condition. P is equal to the actual grate perimeter in feet (three sides) subtracting out length occupied by bars and disregarding the side against the curb.

$$Q_i = C_w P_e d^{1.5} \qquad (5.22)$$

$$P_e = (1 - C_L)P \qquad (5.23)$$

$$P = \frac{Q_i}{C_w d^{1.5}(1-C_L)} = \frac{13\,cfs}{3.0 \times 0.32^{1.5}(1-0.5)} = 47.8\,ft$$

Step 3:

Substitute Equation (5.25) into Equation (5.24) to solve for grate length in orifice condition.

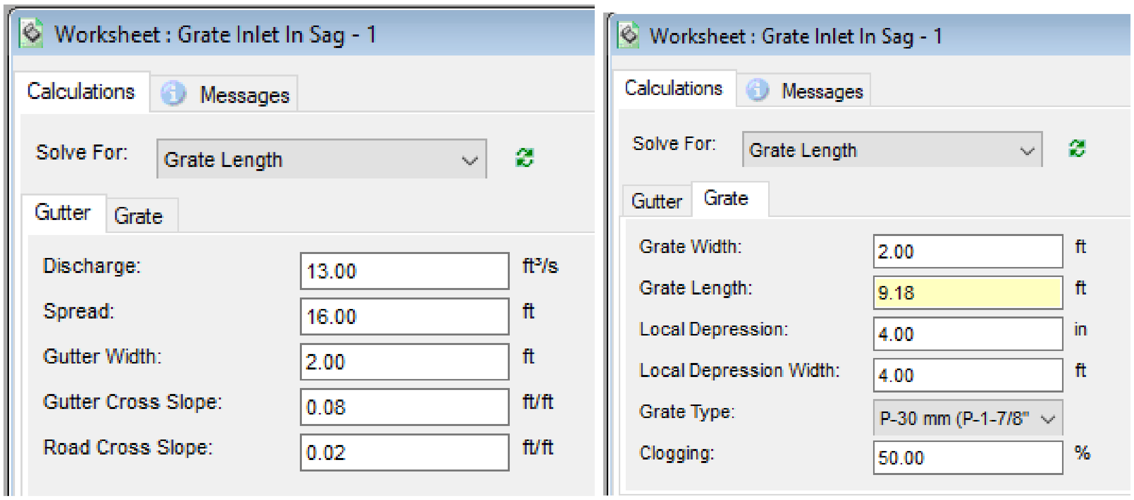
$$Q_i = C_o A_e (2gd)^{0.5} \qquad (5.24)$$

$$A_e = (1 - C_L)A \qquad (5.25)$$

$$L_{Grate} = \frac{Q_i}{C_O(2gd)^{0.5}(1-C_L)W_{Grate}} = \frac{13}{0.67(2g0.32)^{0.5}(1-0.5)2.0} = 4.3'$$

Since the RCFC Standard Grate length is 3.36 feet, use two grates end to end to exceed the 4.3 feet calculated.

Example 6 (FlowMaster Software)



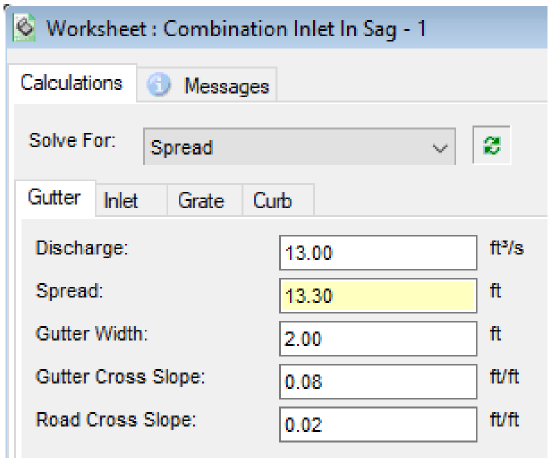
5.3.8 Design Example 7 – Sump Combination Curb and Grate Inlet

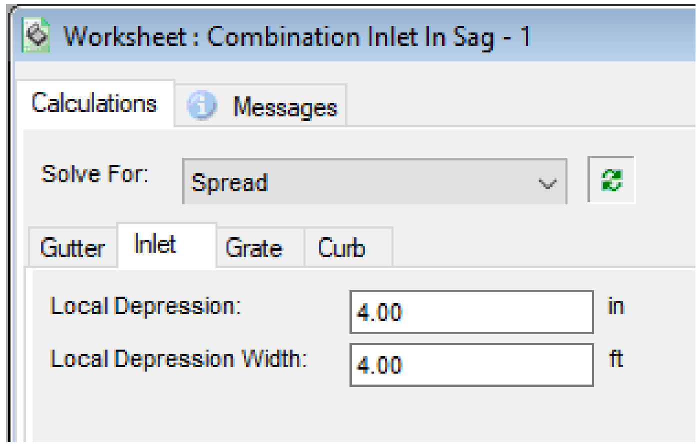
Determine length of a combined curb and grate inlet in a sump condition required to maintain the ponding conditions at the acceptable depth previously determined in Example 5.

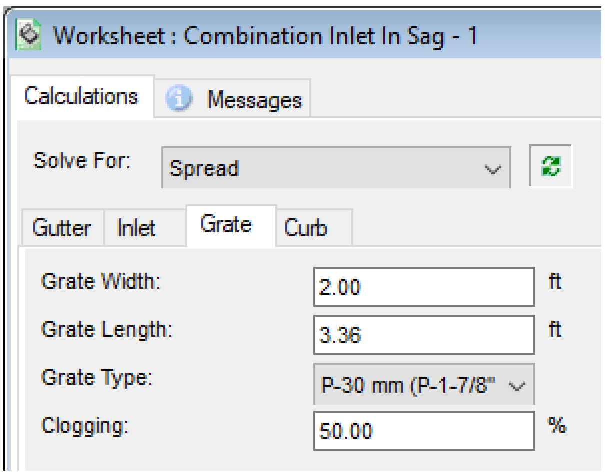
Given:

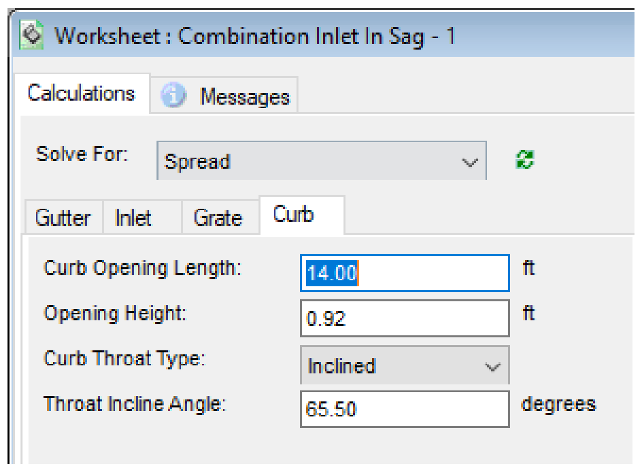
Design flow on south side of South Hampton Ct	Q	=	13 cfs
Cross slope	S_X	=	0.02 ft/ft
Allowable spread	*T*	=	16 ft
Gutter depression at inlet	a	=	0.45 ft
Width of gutter at inlet (local depression)	W	=	4.0 ft
Weir coefficient	C_W	=	3.0
Orifice coefficient	Co	=	0.67
Grate type (see Table 5.3 RCFC)		=	P-30mm (P-1-7/8")
Grate dimensions (see Table 5.3 RCFC)		=	3.36 ft by 2 ft
Clogging factor (standard for all grated inlets)	C_L	=	50 percent
Gutter Width		=	2 ft
Curb Throat Type (Standard for RCFC C.B.)		=	Inclined
Throat Incline Angle		=	65.5°

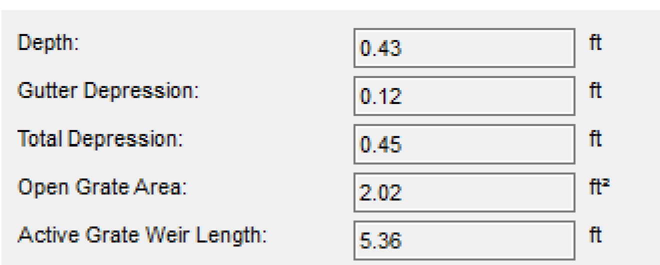
Example 7 (FlowMaster Software)











The depth of 0.43' is less than the previously determined accepted depth of 0.44' so a Combination Inlet with a 14' Catch Basin Opening and 1 (2' x 3.36') grate should be selected to eliminate the hazardous ponding conditions on South Hampton Court.

5.3.9 Design Example 8 – Catch Basin Sizing

This example problem is valid for both on-grade and in-sump conditions. Note that CIVILD will assume in-sag conditions if the longitudinal street slope is less than 0.5%.

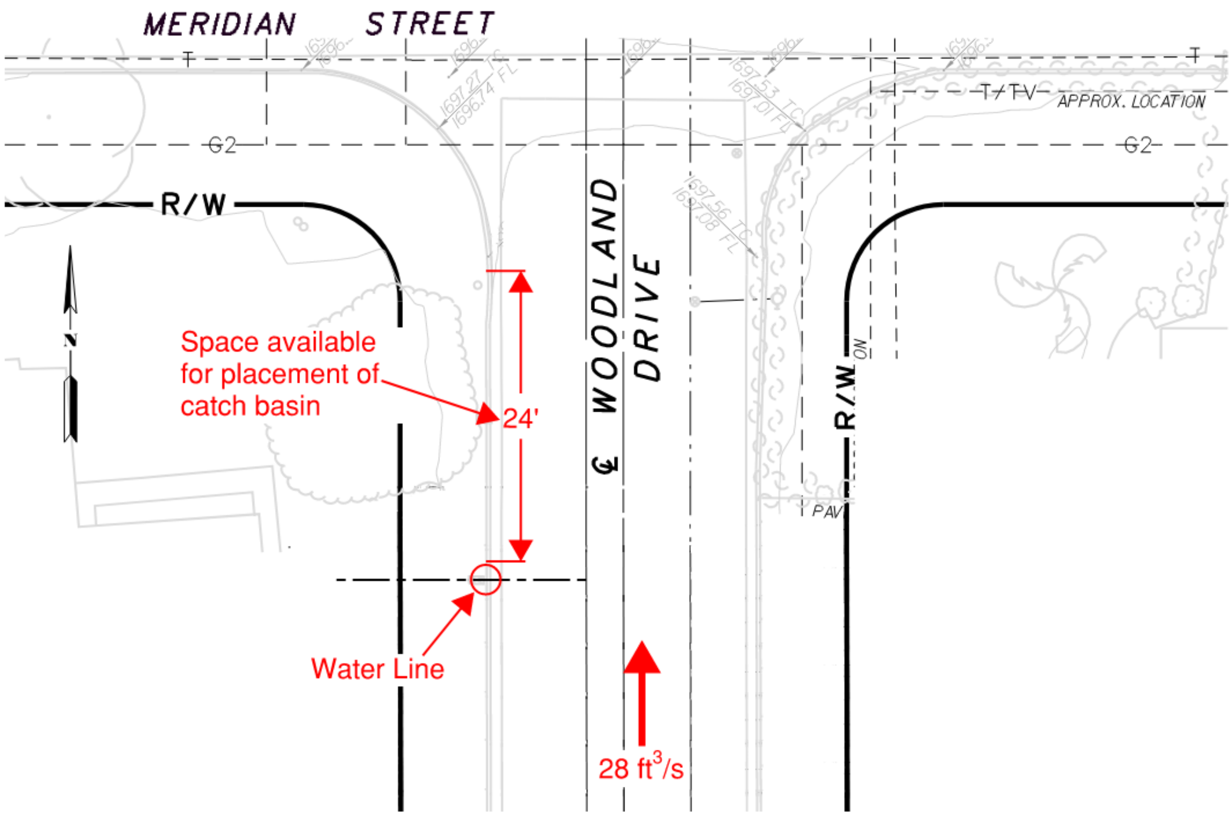
Problem: From a 100-year hydrology study of the area it is known that 28 cfs is traveling down Woodland Drive. Determine the amount of flow traveling on the west side of Woodland Drive.

Calculate the curb opening inlet length that will capture the flow, but also fits in the space available.

Given:

Discharge	Q =	28 cfs
Longitudinal slope	S =	0.005 ft/ft

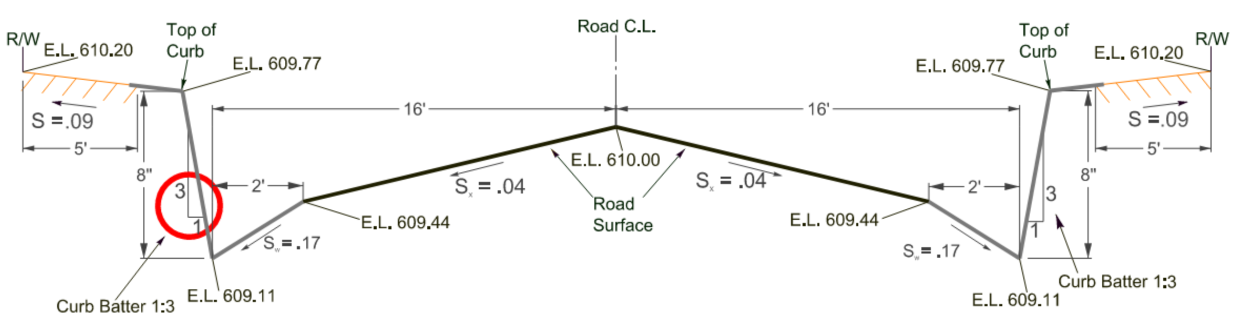
PLAN



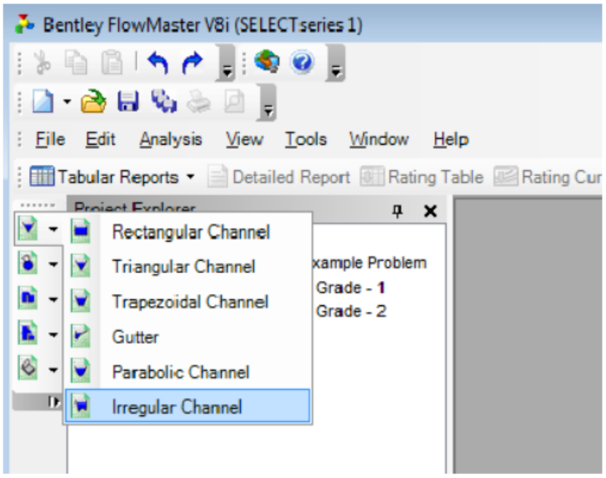
Step 1 (using FlowMaster software):

Determine the amount of flow on the west side of Woodland Drive.

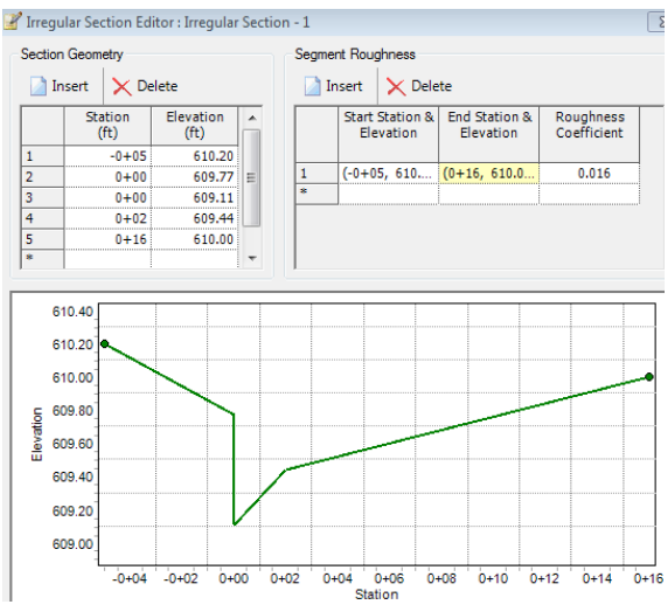
Cross-Section of Woodland Drive



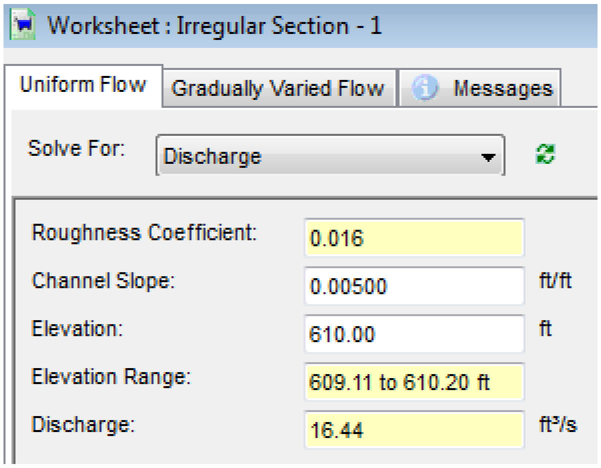
Use the 'Irregular Channel' Application in FlowMaster to determine the capacity on the west side of the street.



Select the Edit Section button in the Irregular Section dialogue window and enter in geometry that represents the cross-section for the west side of Woodland Drive, based on topographic (DTM) data (preferred) or other sources.



Solve for the capacity of the cross-section; in this example the elevation (in dialogue box below) was selected assuming flow depth does not exceed the crown.

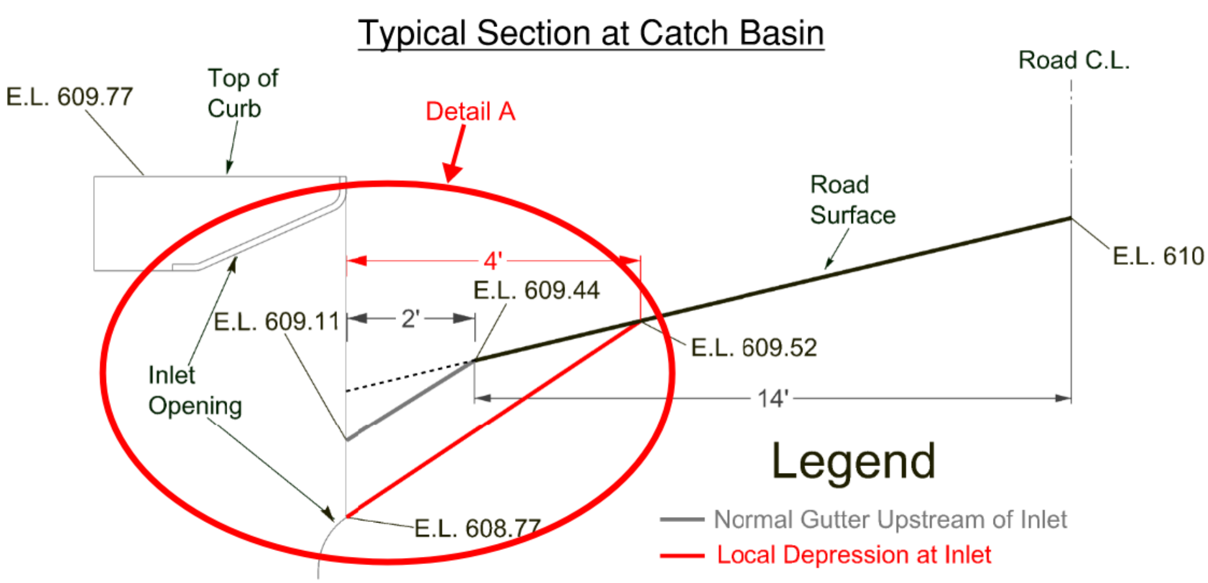


From the dialogue window the capacity of the west side of Woodland Drive is 16.44 cfs.

Note: The side of the street that will reach capacity first must be considered. For this example, it was assumed that the west side of Woodland Drive would fill up first, meaning that 16.44 cfs will flow on the west side and the remaining 11.56 cfs will spill over the crown to the east side. Since the street cross section at this location was verified to be symmetrical, the east side of the street would flow at a depth below the crown. If the total flow was greater than the capacity for both sides of the street, the water level will rise higher than the road crown and eventually raise to the property lines behind the top of curb.

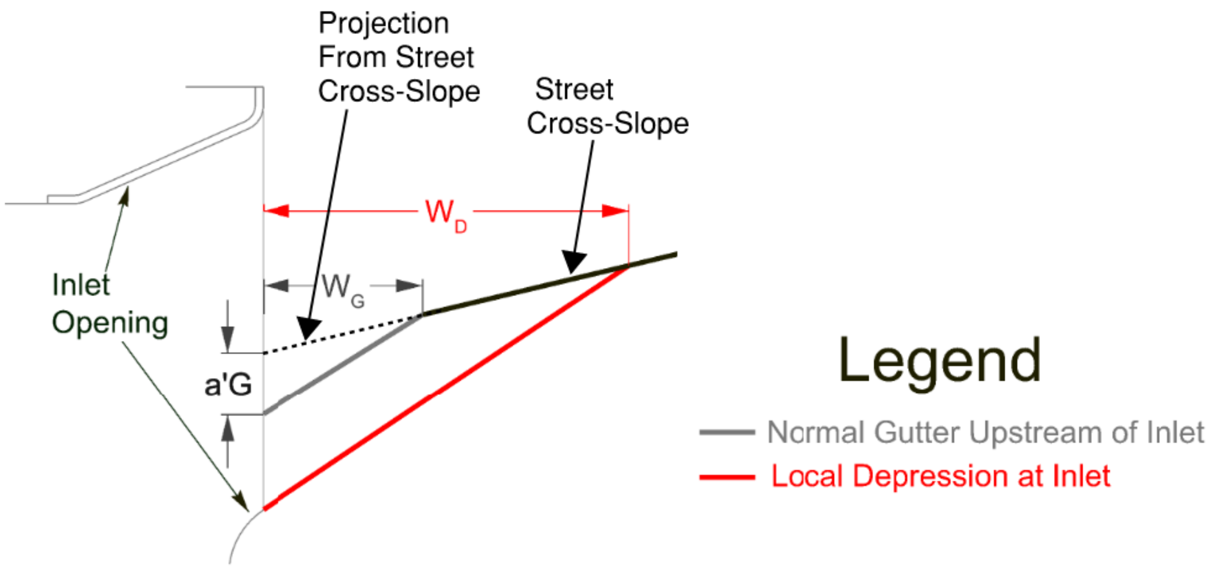
Step 2 (using CivilD software):

Determine the curb opening inlet length required for 100% flow capture on the west side of Woodland Drive.



The gutter hike in CivilD is defined as a'G. The gutter hike is the distance between the projected street cross-slope and the flowline of the gutter.

Figure 5.12: Detail A



Launch CivilD Hydrology/Hydraulics Programs, select Option 5 General Hydraulics (Street, Inlet). Create or modify study file using Option 4 (Street flow + Street Inlet Analysis). Enter maximum flowrate at headworks = 16.44 cfs.

CivilD Input	Variable	Description	Value
Flow Rate			
Enter maximum flow rate at headworks	Q	Design flow rate tributary to catch basin	16.44cfs
Longitudinal Slope			
Enter <system headworks> invert elevation	-	Enter flowline elevation upstream of catch basin	610.1' (select value to get S_L = 0.005)
Enter <system outlet> invert elevation	-	Enter flowline elevation downstream of catch basin	610.0' (select value to get S_L = 0.005)
Enter length or distance between above points	-	Enter length between the two flowline elevations	20' (select value to get S_L = 0.005)
Street Geometry Data Entry			
Enter curb height above gutter flowline	H_c	Height of curb upstream of catch basin	8" (from cross-section of Woodland Dr)

CivilD Input	Variable	Description	Value
Enter the distance from curb to property line	D_{back}	Distance from curb face to property line	5' (from cross-section of Woodland Dr)
Enter width of half street from curb face to crown	D_{half}	Centerline to curb face	16' (from cross-section of Woodland Dr)
Enter distance from street crown to grade break	D_{break}	If not applicable enter distance ≤ D_{half} - W_D	12' (since no grade break present, 12' = street half-width minus local depression width)
Enter the gutter width	W_G	Width of gutter upstream of catch basin	2' (from cross-section at catch basin)
Enter the gutter hike	a'G	See Figure 5.8 and Figure 5.12	2"
Enter the slope <vert/horiz> from curb to property line	S_{back}	Slope from curb face to R.W.	.086 (from cross-section of Woodland Dr)
Enter slope from grade break to crown	S_X	Street cross slope	0.04 (from cross-section at catch basin)
Enter slope from curb gutter to grade break	S_X or S_{X1}	If not grade break, enter same value as street cross slope	0.04 (from cross-section at catch basin)
Select streetflow option desired	-	Follow program guidance	
Curb Inlet Data Entry			
Enter the inlet length	L	Either 7', 14', 21' or 28'	14'
Enter the width of street depression (at catch basin)	W_D	Typically, 4' per TLMA Std No. 311 or RCFC Std No. LD201	4'

CivilD Input	Variable	Description	Value
Enter the depth of street depression	a'D	Typically, 3" per TLMA Std No. 311, See Figure 5.8. Example uses 4" per previous standard.	4"
Enter the height of curb inlet	h	See Figure 5.8; and Table 5.2 for inclined throat; h=[(h'c+a'D)/cos(24.5°)]-β, where h'c = 0.667', a'D=4"	0.95'

Create an OUT file to view the results.

```
Half street cross section data points through curb inlet:
              X-coordinate (Ft.)    Y-coordinate (Ft.)
            0.0000                  1.4300 right of way
            5.0000                  1.0000 top of curb
            5.0000                  0.0000 flow line
            9.0000                  0.5000 gutter/depression end
            9.0000                  0.5000 grade break
           21.0000                  0.9800 crown
 Length required for total flow interception = Lt
 Lt = .6 * Q^0.42 * Slope^.3 * (1/(n*Se)^.6 =     19.333(Ft.)
 where Manning's n = 0.0160 and Slope = street slope =  0.0050
 Se = Equivalent street x-slope including depression =  0.0961
 Efficiency = 1 - (1-L/Lt)^1.8 =  0.9016
```

The curb opening inlet length required to capture the flow is 19.33 ft so this would be upsized to a 21' catch basin. From the original problem statement, there was 24' of available space for the catch basin, so a 21' catch basin will fit in the space available.

Note: CivilD would assume a Sump condition if the longitudinal street slope of Woodland Drive was less than 0.5%.

5.4 WATER QUALITY TREATMENT

For the Santa Ana Region of Riverside County, projects that involve the construction of new transportation surfaces or the improvement of existing transportation surfaces (including Class I Bikeways and sidewalks), LID-based BMPs may be incorporated into the project design, as feasible. The Engineer is directed to Santa Ana Region MS4 Permit Program Template for Low Impact Development: Guidance and Standards for Transportation projects (https://rcwatershed.org/watersheds/middle-santa-ana-river-watershed/#83-108-04-exhibit-d) and the Design Handbook for Low Impact Development Best Management Practices for the Santa Ana Region of Riverside County (https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/#93-98-1-lid-bmp-design-handbook).

For the Santa Margarita Region of Riverside County, projects that involve new or retrofit paved sidewalks, bicycle lanes, or trails, or the retrofit or redevelopment of the existing paved alleys, streets, or roads may incorporate LID features when feasible within the constraints of existing roadways. The Engineer is directed to the Water Quality Management Plan for the Santa Margarita Region of Riverside County (https://rcwatershed.org/watersheds/santa-margarita-river-watershed/smrwma-clearinghouse/) and the US EPA Green Streets Handbook, March 2021 (https://www.epa.gov/sites/default/files/2021-04/documents/green_streets_design_manual_feb_2021_web_res_small_508.pdf) and the Design Handbook for Low Impact Development Best Practices for the Santa Margarita Region of Riverside County (https://rcwatershed.org/permittees/riverside-county-lid-bmp-handbook/#93-98-1-lid-bmp-design-handbook).

Where pollutant control water quality BMPs are incorporated, the pollutant control BMP capacity shall be additive to the 10-year and 100-year flood capacities.

5.5 SAFETY

In addition to providing for traffic circulation, streets are also used to convey stormwater. Excessive stormwater depths threaten safe vehicular passage, including passage of emergency vehicles. Whenever grated catch basins are specified, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, and structural adequacy.

5.6 *REFERENCES*

5.6.1 Cited in Text

County of Riverside, Transportation Department, Improvement Plan Check Policies and Guidelines https://trans.rctlma.org Plan Check Guidelines

County of Riverside, Ordinance No. 460 Regulating the Division of Land of the County of Riverside https://rivcocob.org/sites/g/files/aldnop311/files/migrated/wp-content-uploads-2009-10-Final-Ordinance-No.-460.pdf

County of Riverside, General Plan Safety Element https://planning.rctlma.org/sites/g/files/aldnop416/files/migrated/Portals-14-genplan-2021-elements-Ch06-Safety-092821.pdf

U.S. Department of Transportation (USDOT), Federal Highway Administration, 1984, *Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements*. [https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec12.pdf] Publication No. FHWA-TS-84-202

——, August 2013, *Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual*. [https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf] FHWA-NHI-10-009.

5.6.2 References Relevant to Chapter

American Public Works Association, 1981, "Urban Storm Water Management." Special Report No. 49.

American Society of Civil Engineers, 1992, *Design and Construction of Urban Stormwater Management Systems*. ASCE Manual of Practice No. 77/ WEF Manual of Practice No. FD-20. New York.

County of Riverside, 2007, Ordinance No. 461 *Road Improvement Standards and Specification.*

King & Brater, "Handbook of Hydraulics," Section Four, Fifth Edition.

U.S. Department of Transportation (USDOT), Federal Highway Administration, August 2001, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, 2nd Edition. FHWA-NHI-01-020.

6 UNDERGROUND STORM DRAINS

6.1 INTRODUCTION

This chapter describes the methodology and general design criteria that is accepted by the District for the hydraulic design of a storm drain system. In this manual, a storm drain (or storm drain system) refers to coordinated underground conduits (pipes or box culverts), manholes, and various other structural appurtenances designed to convey stormwater runoff to a point of discharge. Underground conduits should be designed to operate in conjunction with surface or street drainage to maintain public safety and manage flooding during storm events and are typically required to have the capacity to convey the peak discharge from a 100-year design event (i.e., design storm) without adversely affecting property located adjacent to the right of way. Street drainage systems shall meet the criteria regarding the maximum flow width, depth, and velocity as described in Chapter 5 of this manual.

The designer of the storm drain system is expected to evaluate and solve issues related to conflicts that can occur with other underground utilities, right of way, access, and maintenance of the system. When the designer must deviate from the requirements of this chapter, they should contact the governing agency as soon as possible to explain the situation and agree upon an acceptable solution.

6.2 GENERAL DESIGN CRITERIA FOR STORM DRAINS

6.2.1 Hydraulic Capacity and Size

Underground storm drains conveying flow within the right of way of a public road, or across a public road (i.e., from one side to the other, across the centerline of a roadway) are typically required to have the capacity to convey the peak discharge from a 100-year design event. However, governing agencies may require that additional or alternate storm frequencies be evaluated based on individual project circumstances, such as local requirements, private roads, sump locations, steep slopes, etc.

6.2.1.1 Size

At a minimum, storm drains within the public right of way shall not be less than 24 inches in diameter or width for mainlines and 18 inches in diameter for laterals. The District will only maintain storm drain pipes larger than 36 inches. The cross-sectional area of the conduit shall not decrease when proceeding down gradient within the storm drain system.

This manual references its design criteria and procedures to storm drain conduit with a circular cross-section. These criteria and procedures can be adapted to other cross-section shapes (e.g., arches, rectangular, or other non-circular shapes) with due care. It is important to note that cross-section shapes must be compared using their section factor ($AR^{2/3}$) where A is cross sectional area of flow and R is hydraulic radius, and not simply based on cross-sectional area and perimeter.

6.2.1.2 Freeboard

For supercritical flow, it may be appropriate to have 1 foot of freeboard in the storm drain conduit. If a hydraulic jump occurs and causes the system to seal, the wetted perimeter and losses increase (significantly for RCB) which will increase the water surface elevation at the manholes, catch basin inlets, etc. potentially making the system deficient to contain the design flows.

The conduit shall convey the design flow with the hydraulic grade line (HGL) maintaining a minimum freeboard of 1.0 ft below the lowest ground surface or gutter flow line (above the conduit) during the design event. The design hydraulic grade line (HGL) shall be at least 0.5’ below the local depression within catch basins.

6.2.2 Allowable Materials

Materials to be used for construction of a public storm drain system to be maintained by the District include:

- Pre-cast Reinforced Concrete Pipe (RCP)
- Cast-in-place Concrete Pipe, if the project geotechnical report supports use of this type
- Reinforced Concrete Box (RCB), either pre-cast or cast-in-place

Storm drains serving as the low-level outlet to jurisdictional dams may include additional requirements such as gasketed connections. Such requirements are beyond the scope of this manual and must be coordinated closely with the local jurisdiction.

Other private or public storm drains that will not be maintained by the District, may be able to utilize a wider variety of materials at the discretion of the local jurisdiction, on a case-by-case basis. These materials may include the listed above concrete conduits, corrugated steel pipe, polypropylene, corrugated aluminum pipe, high-density polyethylene, and other materials. Any system proposed to be dedicated to the District shall however be limited to the materials acceptable to the District.

The selection of pipe material shall consider factors such as strength of the conduit considering the cover, whether high velocity (20 fps or greater) or abrasive flow is anticipated, bedding and backfill conditions over the lifetime of the conduit, ability to add future connections (for public storm drains), potential for wildfires or other future conditions that could affect the integrity of the conduit, groundwater, anticipated loading, length of sections, ease of installation, 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 or presence of a groundwater table), the reasons shall be clearly presented in the supporting project drainage report (see Section 2.3).

6.2.2.1 Cast-In-Place Concrete Pipe

Guidelines for Cast-In-Place Concrete Pipe are discussed in Section V of Memorandum of

Understanding between Riverside County Transportation Department and District. For structural design purposes, the pipe shall be designed flowing no more than just full unless structural calculations are submitted showing that the pipe can safely sustain the proposed hydrostatic head.

6.2.3 Manning's Roughness Coefficient

Table 6.1 provides recommended Manning Roughness Coefficients for underground conduits. The District accepts concrete conduits (in the various forms) for operation and maintenance.

Table 6.1: Average Manning Roughness Coefficients for Closed Conduits

Conduit Material	Manning's n
District-Accepted Materials	
Cast-In-Place Concrete Pipe (CIPP)	
Smooth wood or steel forms	0.014
Rough wood forms	0.017
Reinforced Concrete Pipe (RCP)	0.013
Reinforced Concrete Box (RCB)	
Precast RCB	0.014
Cast-In-Place RCB	0.014
Non-District Storm Drains	
Corrugated Metal Pipe and Pipe Arch (1/2 x 2 2/3 inch corrugations)	
Plain unlined	0.024
Paved invert	0.020
Spun asphalt fully lined	0.013
Plastic Pipe (HDPE and PVC)	
Smooth	0.010
Corrugated	0.024

6.2.4 Alignment and Curvature

6.2.4.1 Horizontal Alignment

Storm drains shall adhere to a straight alignment or a circular curve of uniform radius within the same run of pipe (i.e., from one manhole, inlet, or other drainage structure to another), or follow the alignment of overlying streets whenever reasonable. The District discourages the use of horizontal angle points in storm drain alignments. Where an angle point is necessary, it is limited to 6 degrees without prior approval from the District. Angle points will often require construction of a collar consistent with District standards. Where practical, storm drains shall run perpendicular to the slope contours, not parallel, in cases where the slope is 20 percent or steeper.

The horizontal alignment of a storm drain system shall be consistent with TLMA Standard No. 817 and maintain the minimum horizontal clearance from potable water mains and sanitary sewer lines. The minimum horizontal distance between the outside of a storm drain and the outside of other wet utilities shall not be less than 5 feet without prior approval by the governing agency. Although Title 22 requirements allow 4 feet as the minimum horizontal distance, the lack of as-built plans in most cases makes having a 1-foot buffer and using 5 feet as the minimum horizontal distance preferred. In curved reaches, the material type, length of pipe segments, and bevel of joints can each limit the curvature of the storm drain.

Storm drain shall not be placed under curb and gutter. Storm drain facilities 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.

6.2.4.2 Junction Angle

When designing the junction of two storm drains, priority shall be given to the larger of the connecting storm drains. A main line or back bone storm drain system is usually a large, reinforced concrete pipe or box which conveys stormwater runoff in public right of way (e.g., street, highway, etc.), parking lot, or easement. A lateral or connector pipe is usually a small, reinforced concrete pipe that conveys stormwater runoff from catch basins or other inlets to the main line storm drain. The angle of confluence (φ) measured from the centerline of the main line to the centerline of the lateral shall always be less than or equal to 90 degrees. Figure 6.1 illustrates the definition of angle of confluence used in this manual.

In all cases, the impact of the junction on the mainline and lateral hydraulics must be calculated as described in Section 4.3.2. If for a chosen junction configuration, the hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence, the junction may need to be redesigned to adjust the lateral or mainline conduit size, or use a smaller angle of confluence. Furthermore, the following limits in Table 6.2 indicating the maximum angle of confluence apply:

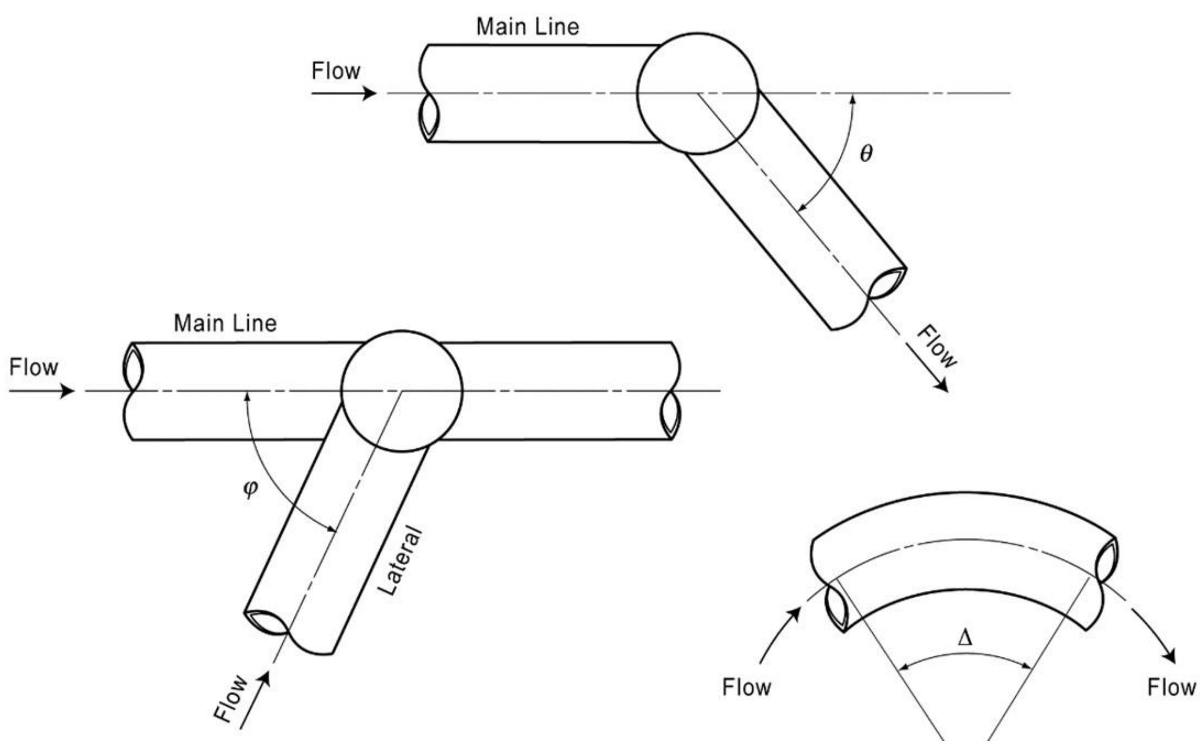
Table 6.2: Maximum Angle of Confluence

Lateral Pipe Diameter	$Q_{LAT} \leq 10\% \ Q_{MAIN}$	$Q_{LAT} > 10\% \ Q_{MAIN}$
24"	90°	60°
30"	90°	60°
36"	60°	45°
42"	60°	45°
48"	60°	45°
54"	60°	45°
60"	45°	45°
66"	45°	30°

Lateral Pipe Diameter	$Q_{LAT} \leq 10\% \ Q_{MAIN}$	$Q_{LAT} > 10\% \ Q_{MAIN}$
72"	45°	30°
78"	45°	30°
84"	45°	30°
≥90"	30°	30°

The angle of confluence shall never exceed 90 degrees. The above requirements in Table 6.2 may be exceeded only if calculations are submitted showing that the use of a larger confluence angle will not unduly increase head losses in the main line and approval by the governing agency.

Figure 6.1: Definition Sketch for Angle of Deflection (θ), Angle of Confluence (φ), and Bend Radius (Δ)



6.2.4.3 Vertical Alignment

The vertical alignment of a storm drain systems shall: (1) maintain a depth of cover sufficient to avoid damage to the facility from overhead traffic and earth loads; (2) minimize conflicts with other underground utilities; and (3) minimize potential buoyancy problems (in cases where a groundwater table is present).

The minimum grade of storm drain conduit shall be 0.003 (.3%). For Cast-in-Place Concrete Pipe (CIPP), the minimum grade shall be 0.005 (.5%) per Section V of District/Transportation MOU. The governing agency may approve flatter grades when sedimentation is not expected and where no other practical solution is available due to topography and/or site constraints. The District does not permit circular curves in the vertical plane for storm drain.

The minimum and maximum soil cover above a storm drain facility depends on storm drain material type and strength, size of conduit, cover material, bedding conditions, and traffic loading. For practical purposes, the range of these conditions cannot be delineated fully in this manual. The designer shall confirm that the design strength of the conduit will be adequate for the design load. For cover less than 2 feet, concrete slurry (2,000 psi) shall be used per District Standard Drawing M815. When there is more than 20 feet of soil cover (measured from top of pipe to finish ground), special design conditions may apply (see "Maintenance Easements ," Section 6.2.9 and "Deep Manholes" Section 6.2.5.5).

When change in vertical grade exceeds 0.1 ft/ft, a concrete collar should be used per District Standard Drawing M803.

Best design practice for the vertical alignment within manholes, junction structures, or equivalent drainage structures is to provide a minimum 1:12 ratio of fall across the structure. When increasing the pipe diameter or box height in the downgrade direction, the standard practice is to match the crowns (soffits) of the incoming and outgoing storm drain conduits when possible. The designer may vary from this practice in consultation with the governing agency.

Alternative storm drain designs (pre-cast vs. cast-in-place) shall have separate hydraulic and structural calculations and note design parameters on the plans. It should be noted that not only can Manning's roughness values be different for pre-cast and cast-in-place construction (see Section 6.2.3), but pre-cast conduits – particularly for RCB – have haunches that need to be accounted for in the hydraulic calculations. Upon completion of project, the Design Engineer shall revise the construction plans and drainage report to indicate the actual conduit used.

Maintain 1-foot minimum vertical clearance between storm drain and other utility crossings, or specify pipe supports where minimum clearance isn't possible. When a sewer mainline crosses over a storm drain with less than 2 feet of separation, the sewer line is required to be encased in steel or concrete.

Lateral pipe entering a main line pipe storm drain generally should be connected radially (spring line to spring line). Lateral pipes or box entering a main line box structure shall be subject to the requirements of the applicable District Standard Drawing for Junction Structures, current edition, or of the project specific design as calculated and stamped by a registered Civil Engineer.

6.2.5 Manholes and Vaults for Storm Drain Maintenance Access

Manholes and vaults (i.e., cleanouts) are structures that allow access for maintenance of a storm drain facility. A manhole is generally placed in a storm drain system at locations of conduit size/slope change, conduit horizontal alignment change, either the begin of curve (BC) or end of curve (EC), pipe intersections, and at other periodic locations to provide access to the system for maintenance. The designer shall specify manholes at prescribed locations within a storm drain facility, and at specific locations in relation to the horizontal and vertical curvature of a pipe alignment.

Manholes are also placed within catch basins in accordance with District/County standard drawings, however, this section is focused on manholes that are placed specifically for maintenance access to the underground storm drain.

6.2.5.1 Manholes: Standard Drawings

Manholes within public right of way must conform to a standard drawing acceptable to the local jurisdiction. Storm drains that will be District maintained must utilize the applicable District Standard drawings.

6.2.5.2 Manholes: General Location

Manholes shall not be located in street intersections as such locations would require closing the intersection for access to the manhole.

Manholes 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 (manhole to be located within public easement), or enters or exits a public right of way;
2. When the upgrade conduit has a steeper slope than the downgrade conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.
3. When transitioning to a smaller downgrade conduit due to an abruptly steeper slope downgrade, debris tends to accumulate at the point of transition. It should be noted that the District will generally not approve transitioning to a smaller downgrade conduit for systems that the District will own. Contact the local jurisdiction before proposing such a transition.
4. Manholes must be placed at regular intervals along the storm drain alignment as prescribed in Table 6.3.

Table 6.3: Maximum Manhole Spacing

Pipe Diameter	**Maximum Manhole Spacing**	
	V < 5 fps	**V ≥ 5 fps**
≤30"	300 ft [1]	300 ft
<45"	400 ft	400 ft
≥45"	400 ft	500 ft

[1] Where the proposed conduit is less than 30 inches in diameter and the horizontal alignment has numerous bends or angle points, the manhole spacing shall be reduced to 200 feet.

For economic purposes, it is beneficial to co-locate the required ‘interval’ manholes at locations as described in bullets 1-3 above, and where structures are otherwise required such as connections between two storm drains, at changes in storm drain sizes, and grade breaks. Transition and junction structures that do not provide maintenance access directly above from the surface may be used for connections between two storm drains, at changes in storm drain sizes, and along horizontal curves when: 1) the minimum cleanout spacing requirements have been satisfied; and 2) there is a cleanout within 50 feet. Lateral storm drain entering a main storm drain shall be connected in accordance with Standard Plans.

6.2.5.3 Manhole: Horizontal Curves

When determining manhole spacing per Table 6.3 and where feasible for storm drain alignments with short bend horizontal curves, manholes shall be located within 50 feet of the upstream end of all storm drain horizontal curves.

6.2.5.4 Pressure Manholes

A pressure manhole shaft and a pressure frame and cover shall be installed, with prior governing agency approval, whenever the design hydraulic grade line is higher than 1 foot below the ground surface.

6.2.5.5 Deep Manholes

A manhole shaft safety landing shall be installed in accordance with District Standard Drawing MH261 when the manhole shaft is 20’ or greater in depth, denoted as distance "M" on District Standard Drawing MH251 or distance "P" on District Standard Drawing MH253.

6.2.5.6 Vault Access

Vault access may be required to allow for maintenance equipment to be dropped into a reinforced concrete box or pipe when sediment deposition is expected due to:

- Undeveloped watershed upstream without debris basins
- Flat slope and/or low velocities (per Section 6.2.7) in facility
- No access available via inlet/outlet for a considerable distance
- Change in slope gradient from steeper to flatter is greater than 10%
- Vault minimum opening and staging dimensions to allow for access operations shall be reviewed on a case-by-case basis.

6.2.5.7 Manholes for RCB

Manholes shall be constructed in accordance with District Standard Drawing MH253. Manhole shafts for cast-in-place and pre-cast RCB shall be aligned with the interior side of an exterior wall. For multi-cell RCB, access to each cell shall be provided. The steps per District Standard Drawing MH259 shall continue from the manhole shaft down the wall of the RCB to provide access to the RCB invert. Due to the steps along the RCB wall at manhole shafts, laterals connecting to the RCB should be offset and located downstream from the manhole location.

6.2.6 Pipe Anchors

Pipe anchors shall be installed for storm drain pipe with slopes greater than 7 feet vertical interval on all pipe slopes of 5:1 (20%) or steeper. Refer to SPPWC 221-2.

6.2.7 Minimum Velocities

Underground storm drain systems shall be self-cleaning (i.e., velocity in conduit is adequate to clean the storm drain) and the minimum permissible velocity for underground systems shall not be less than 3 fps at the design flow rate of the conduit. If the calculated velocity in a storm drain at the design event (typically 100-year) is 3 ft/s or less and there is a risk of sediment or debris entering the system, the Design Engineer shall confirm that conduit is self-cleaning at various additional low flow rates (such as for the 2-year and 5-year events).

6.2.8 Maximum Velocities and Abrasion

The maximum Velocity allowable within a storm drain will depend on the conduit material. The manufacturer of the conduit should be consulted to determine appropriate limits for the allowable velocity. For concrete conduits used in District storm drains, the maximum velocity allowable without additional mitigation measures is 20 feet per second. The best design practice is to design storm drains that maintain velocities below this threshold. Strategies to reduce velocities within a storm drain can include:

- Reducing slope of the conduit
- Using a larger conduit
- Increasing roughness, such as through the use of velocity rings.

In cases where a storm drain is expected to carry debris or abrasive sediment, or has unavoidable high velocity flows (20 feet per second [fps] and greater), the design shall incorporate added measures to provide sufficient design life for the facility. The concrete coating on the inside of all reinforced concrete pipes and boxes must be increased. As well as the concrete design strength per Table 6.4.

Table 6.4: Concrete Coating and Design Strength with Increased Velocities

Velocity of Flow	Concrete Coating on Inside of Conduit	Concrete Design Strength, f’c (psi)	Concrete Class
Less than 10 ft/s	RCP – N/A RCB – N/A CIPP – per Standard Specifications	4,000	A

Velocity of Flow	Concrete Coating on Inside of Conduit	Concrete Design Strength, f'c (psi)	Concrete Class
Exceeds 10 ft/s	RCP – N/A RCB – N/A CIPP – 140-degree segment of invert shall be thickened 2 inches in wall thickness as "sacrificial concrete"	5,000	A
Exceeds 20 ft/s	RCP – increased to provide a minimum of 1-1/2 inches over reinforcing RCB – increased to a minimum of 3-1/2 inches over reinforcing CIPP- not allowed	5,000	A
Exceeds 30 ft/s	RCP – increased to provide a minimum of 1-1/2 inches over reinforcing RCB – increased to a minimum of 3-1/2 inches over reinforcing CIPP- not allowed	6,000	A
Exceeds 40 ft/s	Shall not be used without prior District approval		

The use of velocity control rings is intended for applications where velocities within the pipe are 20 fps and greater and where no significant bedloads are anticipated, or where other methods of energy dissipation are impracticable. See Section 9.3.3 for additional information regarding velocity control rings.

6.2.9 Maintenance Easements

6.2.9.1 Width

Table 6.5 lists the minimum **storm drain easement** width the District requires for underground storm drain facilities. The required width of easements is determined based upon the following factors.

1. Width required for access with equipment is 10 feet minimum for 18" pipe and

increases with the conduit size and depth.

2. Width required for trenching:
    a. For depths less than 8’, an unshored sloped trench based on Cal-OSHA guidelines Standard Number 1926 Subpart P App B.
    b. Shored trench for depths greater than 8’, assuming trench slopes back at 15 degrees from vertical.
    c. For conditions where other than firm, compacted soil is encountered, shoring for trenches less than 5’ deep will be required.
    d. These are bare minimums for restricted access areas.
3. Storm drains with cover deeper than 25 feet or other special conditions may warrant additional easement width and require agency consultation and approval.

Storm drains and easements shall be placed on one side of lot ownership lines with the edge of easement coinciding with the lot line in new developments. In existing developments, storm drain easements shall follow lot ownership lines to the maximum extent practicable.

6.2.9.2 Exclusivity

It is preferred that storm drain easements be established exclusively for storm drainage facilities, and therefore, no other utilities be located within the easement. If other utilities are proposed within District easements, prior approval from the District must be obtained. Additional requirements, limitations, or exclusions may be imposed by the District to ensure that such utilities do not unduly limit or hinder any required District maintenance.

6.2.9.3 Vehicular and Personnel Access

All storm drain easements through private property shall have physical access from the public right of way to the manholes, vaults, inlets, and outlets. In the event that the storm drain facility cannot be directly accessed from a public road (e.g., a steep slope or grade differential), an access road shall be provided with acceptable grade, vehicle loading, and turning radii for maintenance vehicles and personnel. An **access easement** shall be provided/conveyed to the maintenance entity for this use. For vehicular turn around requirements on facilities to be maintained by the District, see District Standard Drawing M827. See also Section 6.2.12.3.

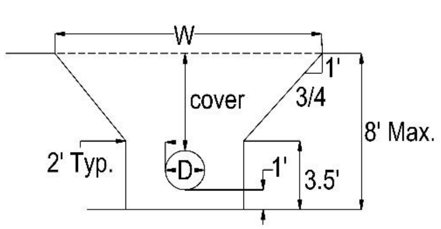
6.2.9.4 Limitations over District Easements

Permanent structural improvements (i.e., buildings, planter boxes, stockpiling earth fill, etc.) shall not be constructed over storm drain easements. If easement area is landscaped, it shall be with ground cover and shrubs, excluding any and all trees and woody vegetation. Facilities such as parking lots, recreation fields and trails, maintenance access roads, and fencing may be approved at the discretion of the governing agency and maintenance entity provided that adequate vehicular access to manholes and inlets/outlets is maintained.

Table 6.5: Storm Drain Minimum Easement Widths

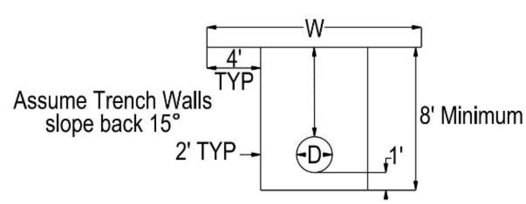
Pipe Dia. (in.)	Cover (feet)													
		Sloped Trench ←					→ Shored Trench							
	2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**12**	**14**	**16**	**18**	**22**	**24**
18	10	10	10	12	14	15	20	20	20	22	22	24	25	28	28
21	10	10	12	12	14	16	20	20	20	22	24	24	25	28	28
24	10	10	12	14	15	20	20	20	22	22	24	24	25	28	30
27	10	12	12	14	15	20	20	22	22	22	24	25	26	28	30
30	10	12	14	15	20	20	20	22	22	24	24	25	26	28	30
33	12	12	14	15	20	20	22	22	22	24	24	26	28	30	30
36	12	14	14	16	20	22	22	22	22	24	24	26	28	30	30
39	12	14	15	16	22	22	22	22	24	24	25	26	28	30	32
42	12	14	15	22	22	22	22	24	24	24	26	28	28	30	32
45	14	15	16	22	22	22	24	24	24	25	26	28	28	30	32
48	14	15	18	22	22	22	24	24	24	25	26	28	28	32	32
51	14	16	18	22	22	24	24	24	25	26	28	28	30	32	32
54	15	16	22	22	24	24	24	25	25	26	28	28	30	32	32
57	16	18	22	24	24	24	25	25	26	26	28	30	30	32	34
60	16	18	24	24	24	24	25	25	26	28	28	30	30	32	34
63	18	18	24	24	24	24	25	26	26	28	28	30	30	34	34
66	18	20	24	24	24	25	25	26	28	28	30	30	32	34	34
69	18	20	24	24	25	25	26	26	28	28	30	30	32	34	35
72	20	24	24	25	25	26	26	28	28	28	30	32	32	34	35
75	24	24	24	25	26	26	28	28	28	30	30	32	32	34	35
78	24	24	25	25	26	26	28	28	28	30	30	32	32	35	36
81	24	25	25	26	26	28	28	28	28	30	32	32	34	35	36
84	25	25	26	26	28	28	28	28	30	30	32	32	34	35	36
87	25	25	26	28	28	28	28	30	30	30	32	32	34	36	38
90	25	26	26	28	28	28	30	30	30	32	32	34	34	36	38
96	26	28	28	28	28	30	30	30	30	32	34	34	35	38	38
102	28	28	28	28	30	30	30	32	32	32	34	34	36	38	40
108	28	28	30	30	30	30	32	32	32	34	34	35	36	38	40
120	28	30	30	32	30	30	32	32	34	35	36	38	38	40	42

SLOPED TRENCH



W=1.54D + .54 cover + 12.54
Round in like manner

SHORED TRENCH



W=1.54D + .54 cover + 12.54
Round in like manner

Note: D is pipe diameter (in feet) or width of RCB.

6.2.10 Transition from Large to Small Storm Drain

Storm drains shall be increasing in the downstream direction. Transitions from large to smaller storm drain conduit diameter/height in the downstream direction, aka telescoping, is prohibited unless an exception is approved by the governing agency and maintenance entity.

6.2.11 Buoyancy

Buoyancy calculations shall be provided under the following conditions:

1. Where the soils report indicates that the maximum groundwater elevation is above the bottom of the storm drain.
2. Where the storm drain is adjacent to levees or other water body that will cause the maximum groundwater elevation to rise above the bottom of the drainage structure.
3. Where the drainage structures are adjacent to stormwater basin or other facility where infiltration of water in the area of the proposed drainage structure is anticipated.
4. Other conditions, which in the judgment of the governing agency and maintenance entity may cause groundwater elevations to rise above the bottom of the drainage structure.

The buoyancy calculations shall be prepared in accordance with the methodology outlined in American Concrete Pipe Association's Design Data 22 (October 2007).

Where buoyancy/flotation is a concern, mitigation measures, such as pipe anchors or other strategies, must be used as approved by the governing agency or maintenance entity.

6.2.12 Inlet and Outlet Considerations

6.2.12.1 Inlet Structures

An inlet structure shall be provided for storm drains located in natural channels. In general, the interception of flows from a natural watercourse directly into a storm drain system shall require an approved inlet structure. The use of a debris or sediment barrier or basin upstream of the inlet structure may be required; see Chapter 11 . The structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron with upstream cutoff wall. The paved inlet apron should be limited to a maximum chute slope of 2:1. Wall heights should conform to the height of the water upstream of the inlet plus freeboard and be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing (chain link or cable rail), an access barrier, and a trash rack to promote public safety should be considered as described in Section 9.7.

6.2.12.2 Outlet Structures

Where conduits or channels discharge into an improved earthen or natural channel, measures must be taken to prevent erosion, headcutting, and property damage. Outlet velocities must be sufficiently reduced to prevent erosion by construction of a suitable energy dissipater (e.g., riprap apron, impact basin, etc.). See Chapter 8 , "Open Channels", for recommended maximum permissible velocities for unlined channels, and Chapter 9 , Section 9.3 for energy dissipation at outlets.

The orientation of the outfall should be pointed in a downstream direction with the receiving channel at the minimum confluence angle allowed. Where a storm drain outfall discharges at an angle to the direction of flow of the receiving channel, consideration should be given to the possibility of erosion on the opposite channel bank, and a channel bank lining of riprap or other suitable material should be installed in the opposing bank. See District Standard Drawing for Junction Structure No. 8 for outlet to earthen channel (JS233).

6.2.12.3 Inlet and Outlet Maintenance Access

For all District maintained pipe or box inlet or outlets, a 15’ access ramp paved with 3" thick of 1" crushed rock (Caltrans gradation for 1" x No. 4 coarse aggregate per Section 90-1.02C(4)(b)), 10% maximum grade, and 2% cross fall, shall be provided to the invert of the inlet and/or the outlet, include a PCC commercial driveway approach, and a pipe swing gate per District Standard Drawing M820 located 35 feet from the edge of traveled way. Access road slopes greater than 10% to a maximum 15% shall obtain pre-approval on a case-by-case basis from the governing agency, and access road shall be paved with grouted Class I Rock (per Caltrans Section 72-3.02C). With prior permission of the governing agency, bollards may be allowed in lieu of the swing gate where the access road also serves as part of a public trail system. A turn around per District Standard Drawing M827 shall also be provided at the inlet and outlet.

On a case-by-case basis, the District may only require access to the top of the inlet and outlet structure where access to the invert of the culvert/storm drain is possible with heavy equipment (parked in a staging area adjacent to the inlet/outlet but outside of vehicle and pedestrian travelled way within a safe and unobstructed reach). The maximum vertical/horizontal distance from the pipe flowline to the staging area shall be less than 10 feet.

6.2.13 Minimum Storm Drain Size

The recommended minimum pipe diameter of publicly maintained storm drains is 24 inches in diameter for mainlines and 18 inches in diameter for laterals. Privately maintained storm drains located in a public right of way shall also be a minimum diameter of 18 inches.

For District maintained reinforced concrete box culverts where there is a potential for debris/sediment in the system, the minimum height shall be 7 feet to allow for use of mechanical equipment to perform maintenance. In addition, maintenance equipment access shafts shall be included every 1,000 feet or at locations specified by the District. Where sediment is unlikely, 4-foot height is acceptable (e.g., built-out watershed or facilities below sediment catchment facilities). The height to width ratio shall be per Caltrans Standard Plan D80 or D81. Crossfall shall be provided for all RCB inverts 8 feet and wider. District’s preference is the alternative labeled "Sloped Invert" per Caltrans Standard Plan D80 or D81.

6.2.14 Storm Drain Plans

Storm drain plans shall be developed consistent with the content, labeling, and other requirements as presented in the District’s Drafting Manual rcflood.org>Business>Engineering Tools>Drafting

Manual, including, but not limited to, the list below. The District's Drafting Manual shall govern if any discrepancies occur with the following list.

- Plan and profile.
- Stationing, which shall increase in the up-grade direction from the downstream end of the storm drain. Stationing shall generally increase from left to right on the drawing sheet.
- Hydraulic Grade Line (HGL), including hydraulic jumps for the design flow rates and any interim condition flow rates.
- Design flow rate in cubic feet per second and maximum velocity in feet per second. (Note: the flow rate that results in the maximum velocity in the conduit may not coincide with design flow rate.)
- Pipe design load rating or equivalent information (depending on pipe material, this might include pipe gauge or wall thickness).
- Where using RCP, the minimum D-Load for 42-inch diameter pipes or smaller is 2,000-D, (Class IV). The District prefers that the D-Load be stated instead of Class for pipes larger than 42 inches.
- Show all existing and proposed utility crossings in profile. If clearance is less than 1-foot, state the clearance on the profile.
- Where high velocities or abrasive conditions exist, provide a note on the profile specifying any additional cover or strength requirements in accordance with Table 6.4.

6.3 HYDRAULIC DESIGN OF STORM DRAINS

Criteria to be used in the estimation of a hydraulic grade line (HGL) for a storm drain are discussed in Chapter 4 . Calculations to determine the hydraulic grade line in a storm drain system flowing subcritical (downstream control) typically begin with a known water surface elevation (WSE) at the downstream end of the system being analyzed, but an assumed WSE may be used with prior approval of the governing agency. In cases where a steep system flowing supercritical (upstream control) is being analyzed, a known downstream WSE is not used when analyzing the system, but instead a known (or assumed) headwater elevation would be utilized.

District-accepted computer applications for hydraulic analysis (e.g., WSPGW, AES, FlowMaster, HEC-RAS) incorporate the theories described below and are encouraged to be used in storm drain facility design. The full list of District-Approved software can be found at (rcflood.org>Business>Engineering Tools>District Accepted SoftwareAccepted-Software.pdf).

Pre-cast RCB will have haunches at the 4 corners that reduce the conveyance area and increases losses. In WSPG, model the geometry with the haunches using the irregular covered card instead of using a standard box size. It is recommended to add a minor loss in the hydraulic model where the cast-in-place transitions to pre-cast.

6.4 HYDRAULIC CRITERIA

6.4.1 <u>Main Line Hydraulic Grade Line</u>

The hydraulic grade of water surface elevations in the storm drain system are determined by using criteria from Chapter 4 . Final design shall be performed using the Water Surface Profile calculations provided in Section 4.4. In some cases, the Engineering Authority may allow the preliminary design to assume normal depth as described in Section 4.2.6.2.

The design shall establish the hydraulic grade line to:

- below the ground surface
- a minimum of 1 foot below manhole rim elevation (unless a pressure manhole is to be utilized
- a minimum of 0.5 feet below the gutter local depression at the catch basin

When the hydraulic grade line rises above ground level, stormwater may discharge out of catch basins or pop manhole covers, either of which can lead to damage, inconvenience to pedestrians and danger associated with vehicular traffic.

Once the main line hydraulic grade line intercepts the elevation of the inflow at a catch basin or other inlet, no further runoff can be admitted to the pipe network. This phenomenon in the field would be evidenced by runoff passing directly over the catch basin/inlet to flow down the street (or overland) until it enters the system elsewhere. Another indication is water standing in sumps (storage facility ponding) until there is sufficient capacity in the storm drain to admit the ponded water.

6.4.2 Determining Controlling Water Surface Elevation

See Section 4.4.3.1 for determining the controlling water surface elevation for hydraulic water surface profile calculations.

6.4.3 Connector Pipe Hydraulic Grade Line

Connector pipes connecting catch basins to storm drains can be sized and/or evaluated by estimating headlosses due to:

- friction over the length of the connector pipe, and
- entrance (inlet) losses where the connector pipe meets the inside wall of the catch basin.

Friction Losses - The downstream HGL used for the connector pipe shall be the peak water surface for the mainline after consideration of the junction. Establishing the actual friction losses in a connector pipe requires a water surface profile analysis in accordance with Chapter 4 . A conservative estimate of the losses may be determined by assuming the pipe is flowing full, which simplifies the calculations.

Entrance Losses - The total headlosses at the inlet of the connector pipe can be estimated by using Equation (6.1). Equation (6.1) is a modified form of Equation (4.37):

$$h_i = (1 + k_{en})\frac{V^2}{2g} \tag{6.1}$$

where:

h_i = headloss at inlet (ft)
k_{en} = entrance loss coefficient

The k_{en} in the equation is equivalent to k_e values listed in Table 7.1.

A concrete connector pipe joining a catch basin constructed in accordance with District/County standards will use k_{en} = 0.2.

6.5 SAFETY

Storm drain inlets and outlets require consideration for both life safety and the debris transported by stormwater. Frequently, trash screening devices (e.g., Greenbook Standard Plan 361) are also used as devices that improve safety by blocking public from entering an underground storm drain. The need for safety devices and trash/debris screening devices at conduit inlets/outlets should be determined during planning, prior to the design of stormwater management facilities.

Protective access barriers at conduit inlets and outlets shall be provided wherever it is necessary to prevent unauthorized public access and potential entrapment during both dry and wet weather periods. Typical public facilities where such structures should be considered include parks, open space, or areas where the public may be present near storm drain inlets and outlets.

It is rare that cost-effective access barriers and trash collectors can be retroactively added without a reduction of intended system design capacity, and therefore **these measures should be included, and their hydraulic impacts analyzed during the planning and design phase**. See Section 9.7 for more information.

The District recommends that appropriate access barriers (such as but not limited to trash racks) be included on the entrances and access barriers on outlets to all conduits or other hydraulic structures **where unauthorized access is a concern, and** that meet the following requirements:

- Areas of high debris or trash potential.
- When a conduit or hydraulic structure outfalls into a channel with side slopes steeper than 4(H): 1(V) for hydraulically smooth (concrete and soil cement) banks, 3(H): 1(V) for riprap linings, 2(H): 1(V) gabion embankments, and 1(H): 1(V) stepped side slopes.
- Conduits and hydraulic structures with a cross sectional area of 20 square feet or less.
- Conduits and hydraulic structures with a cross sectional area greater than 20 square feet and longer than 200 feet in length.
- Conduits and hydraulic structures with energy dissipaters at the end (Section 9.3).
- Conduits and hydraulic structures being used as inlets or outlets to/from multiple-use facilities such as detention basins or channels that have parks or trails facilities integrated into them.
- Pipes or box culverts and hydraulic structures with enough curvature that the opposite ends cannot be clearly seen.

6.6 REFERENCES

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6.6.2 References Relevant to Chapter

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7 CULVERTS & BRIDGES

7.1 INTRODUCTION

Culverts are hydraulically short conduits (typically pipe, pipe-arch, or box) that convey stormwater under roads, railroad embankments, or other obstructions. Bridges are similar to culverts in their ability to convey water through a crossing. The intent of this chapter is to provide guidance for the design of culvert and hydraulic analysis of bridge openings. Culvert design guidance includes hydraulic analysis, location and alignment, debris loading, channel/culvert stability and sediment movement, maintenance requirements, treatment of inlets and outlets, safety, structural considerations, and economic, and life-cycle costs. The design of bridges requires special training and experience regarding hydraulic analyses, design of flow training works, and estimates of pier and abutment scour. Therefore, only an overview of the hydraulic analyses for bridge openings is presented.

7.2 CULVERTS

The charts and procedures for culvert design used in this manual are taken from the *Federal Highway Administration, Hydraulic Design Series Number 5, Hydraulic Design of Highway Culverts* (USDOT, FHWA, HDS-5, 2012).

7.2.1 Use of Culverts

Culverts are primarily used for conveying runoff through a roadway embankment. They are normally aligned with a watercourse or engineered drainage channel. Due to their low cost and flexibility to be either prefabricated or constructed onsite, culvert crossings are typically used where hydraulics, geometry, and environmental considerations permit. This is frequently the case where smaller watercourses cross a roadfill. Where use of a culvert is not practical due to these considerations, bridges may be a better choice. This is typically the case where larger washes or rivers cross a roadfill.

The District typically does not accept road culverts for maintenance unless the road crossing is part of a broader District-maintained engineered system upstream and/or downstream.

7.2.2 Culvert Design Criteria

7.2.2.1 Sizing and Slope

Minimum culvert sizing shall be 18 inches diameter round pipe and minimum longitudinal slope shall be 0.003ft/ft (0.3%) for facilities within the public right of way. Jurisdictions other than Riverside County may have different requirements.

7.2.2.2 Reinforced Concrete Box (RCB) Culverts: Height, Width, and Crossfall

The height to width ratio of box culverts shall be per Caltrans Standard Plan D80. Crossfall shall be provided for all RCB inverts 8 feet and wider. Crossfall shall be per Caltrans D80 or D81. The

District's preference is the alternative labeled "Sloped Invert".

Where sediment is unlikely, 4-foot height is acceptable for District facilities (e.g., built-out watershed or facilities below sediment catchment facilities). Where there is the potential for debris/sediment in the system, the minimum height shall be 7 feet to allow for use of mechanical equipment to perform maintenance. Whereas, for Riverside County Transportation maintained box culverts or storm drains, the minimum height of 6 feet may be required for maintenance purposes due to CALOSHA requirements per *Improvement Plan Check Policies and Guidelines* (TLMA, 2015) Section J Item 26. Other heights may be allowed as approved by the local governing agency.

Pre-cast RCB will have haunches at the four corners that reduce the conveyance area and increases losses. In WSPG, model the geometry with the haunches using the irregular covered card instead of using a standard box size. It is recommended to add a minor loss in the hydraulic model where the cast-in-place transitions to pre-cast.

7.2.2.3 Minimum Velocity

A minimum velocity of 3 fps for 100-year flows shall be provided. Greater velocities are recommended for installations where sediment loads are heavy. Alternatively, a debris catchment /sediment trap upstream of the culvert can be utilized where culvert velocities are lower or excessive debris or sediment deposition is expected. See discussions in Section 9.7 and Section 11 . Culverts should be designed to provide adequate velocity to self-clean during partial depth flow events. Debo and Reese (1995) suggest a minimum velocity of 2.5 feet per second for partial flow depths.

7.2.2.4 Maximum Velocity

Culvert velocities in excess of 20 ft/s shall require special design, such as increased concrete cover over the reinforcing steel, or increased aggregate size along the invert. See discussion in Section 6.2.8. Outlet velocities must be checked and managed in accordance with Section 7.3.4.

7.2.2.5 Materials

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability, availability, cost, and corrosion and abrasion resistance. There are a wide variety of materials that may be used for construction of culverts, including: pre-cast and cast-in-place concrete, corrugated steel, high density polyethylene (HDPE), polyvinyl chloride (PVC), as well as other materials. The specified material shall be approved by the governing agency. In the unincorporated County of Riverside, the material shall meet the design criteria outlined in Riverside County Transportation Department *Improvement Plan Check Policies and Guidelines* (TLMA, 2015) Section IV.J and Appendix A14. Where field conditions dictate the use of one material over others (e.g., corrosive soil conditions, presence of groundwater table, fire prone areas, or vandalism), the reasons shall be clearly presented in the supporting hydraulic study. The District typically does not accept road culverts for maintenance unless the road crossing is part of a broader District-maintained engineered system upstream and/or downstream. Where culverts connect to District facilities, the material of the entire culvert shall be reinforced concrete.

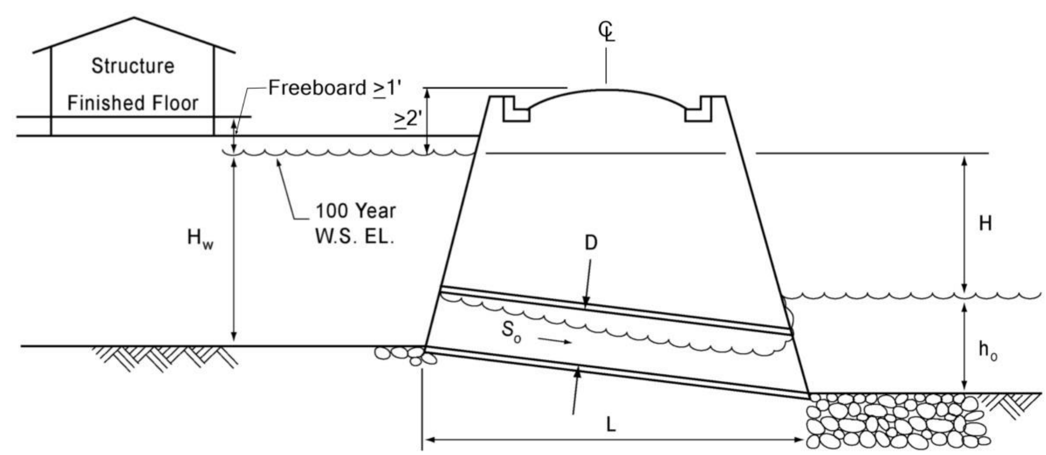
7.2.2.6 Minimum Cover

Minimum cover over some types of culverts must be provided in order to maintain the integrity of the structure under anticipated loading conditions. For example, pre-cast concrete pipe culvert manufacturers may provide minimum cover requirements to achieve the specified D-Load. Whereas, with cast-in-place concrete box culverts designed to Caltrans standards, the culvert may directly receive wheel loads. Cover shall be based on an evaluation of dead and live loads to provide adequate structural protection. Generally, the top of culverts should not extend into the roadway subgrade unless the roadway design has considered this situation. Where cover over pipe culverts is less than or equal to two feet and the culvert is to be maintained by the District, the Flood Control District requires a concrete slurry backfill over the pipe per District Standard Drawing M815.

7.2.2.7 Freeboard

Culvert headwater elevations shall maintain a freeboard of at least two feet below road centerline profile grade and one foot below 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 appropriate documentation (e.g., drainage easement) from all affected property owners as required by the governing agency.

Figure 7.1: Definition Sketch for Culverts



7.2.2.8 Upstream Headwater Ponding

Flows up to and including the 100-year frequency event shall not cause increased flooding to adjacent property or buildings unless a drainage easement is acquired for those areas. Within the supporting drainage study, the ponded headwater elevation shall be delineated on a contour map, or other surveying methods used to identify the area inundated by the ponded water.

7.2.2.9 Roadway overtopping

Road culvert crossings shall be designed to convey the 100-year storm runoff **without** overtopping the road where possible. In cases where overtopping cannot be avoided and with prior approval of the governing agency:

- the overtopping depth shall be less than 9 inches
- velocity less than 1.5 fps
- overtopping flows shall be directed to an area downstream of the crossing to which the flow would have gone in the absence of the crossing

In general, low-water crossings (also known as dip crossings, at-grade crossings, or "Arizona crossings") are hazardous in storm conditions and generally not recommended. However, for flows crossing broad shallow washes where the construction of a culvert or bridge is not practical, the road may be dipped to allow the entire flow to cross the road. Use of dip sections for specific, individual cases must be pre-approved by the governing agency. The pavement through the dip section should be concrete and should have a one-way slope in the direction of flow with curbs and medians flush with the pavement. Upstream and downstream cutoff walls and aprons should be provided to minimize the effects of headcutting and erosion. Signage for travel in both directions warning of the low-water crossing shall also be provided a sufficient distance (per local agency requirements) prior to reaching the crossing.

7.2.2.10 Abrasion and Deposition

An effective culvert design carefully manages velocities in the upstream approach, through the culvert, and across the downstream outlet, considering the associated risk for both scour and deposition.

When a culvert is anticipated to receive significant amounts of debris (such as in a natural tributary that may be subject to wildfire) or sediment (where a natural erodible wash or watershed is upstream), the designer shall consider if a debris basin or sediment trap is warranted upstream of the culvert to ensure the culvert is protected from both abrasion and deposition. See Section 11 as well as Section 9.7 regarding trash racks when applicable.

Where such a feature is not feasible or desirable, the culvert must be designed in a manner to ensure that the debris/sediment does not block or accumulate within the culvert and does not damage the culvert due to abrasion. There are three elements that must be checked and designed carefully.

1) Upstream approach velocity

The designer shall maintain or accelerate the velocity of flow approaching the culvert rather than creating a pond at the immediate entrance, and the design flow shall be increased by an appropriate bulking factor in the hydraulic calculations to account for the debris.

2) Culvert velocity and abrasion

Within the culvert, the following criteria must be met:

Minimum Velocity See Section 7.2.2.3

Maximum Velocity	See Section 7.2.2.4
Abrasion Mitigation	See Section 6.2.8

3) Outlet / tailwater conditions

In areas with steep gradients and heavy debris/sediment loads, full use of the available head may develop excessive velocity resulting in abrasion of the culvert itself and/or downstream scour. A larger culvert operating at a lower velocity, or use of other design strategies to reduce culvert velocities may be required to increase the culvert invert service life. Reducing culvert velocities may help decrease the energy dissipater footprint downstream of the culvert. See also Section 6.2.7. Maintenance access shall be provided to the upstream culvert where multi-barrel installations are utilized due to the increased likelihood of deposition from the ponding upstream of the culvert.

For outlet condition, consider tailwater/backwater effects within the culvert that can slow water down within the barrel and cause deposition. This is not due to barrel velocity alone being too slow, but due to tailwater effects.

To minimize sedimentation problems, depression of the upstream culvert invert below the natural channel flowline should be avoided. But where a culvert invert must be depressed below the natural stream elevation, a cutoff wall shall be constructed at the grade break from the natural stream to the sloping section upstream of the culvert to mitigate against headcutting. Additionally, the slope from the grade break to the culvert inlet must be less than or equal to four horizontal to one vertical and provide reasonable access to facilitate maintenance crew access to the culvert, approved by the governing agency.

7.2.2.11 Skewed Culverts

A good culvert design is one that limits the hydraulic and environmental stress placed on an existing natural watercourse. This stress can be minimized by designing a culvert that closely conforms to the natural stream in alignment and grade. Often the culvert barrel must be skewed with respect to the roadway centerline to accomplish this goal.

The alignment of a culvert barrel with respect to a line perpendicular to the roadway centerline at the point of crossing is referred to as the barrel skew angle (Figure 7.2). A culvert aligned normal (perpendicular) to the roadway centerline has a zero-skew angle. Some advantages of following a natural stream alignment include:

- minimizing headwater at culvert entrance by avoiding angle point losses
- decreasing sedimentation at the upstream end by maintaining approach velocities
- shorter culverts resulting in decreased grading and materials costs

When the culvert barrel is skewed in relation to the road, the face of the inlet and/or outlet may also be skewed in relation to the barrel. This is referred to as the inlet skew angle (Figure 7.3). Inlets may be skewed so that headwalls can be constructed parallel to the roadway and, therefore, avoid warping of the embankment fill slope. With skewed inlets and outlets, the structural integrity of circular sections may be compromised due to the loss of a portion of the full circular section where the culvert barrel is modified to achieve the skew. Where box sections are utilized, a skewed inlet or outlet will increase the distance which the box lid must span, and therefore, a thickened concrete section and/or increased reinforcing steel may be required. Although concrete headwalls help stabilize these sections, structural considerations must not be overlooked in the design of skewed inlets.

In cases where the culvert barrel cannot be aligned with the channel flowline, the culvert barrel centerline should not be greater than 90 degrees from the approach channel flow direction, see Figure 7.4. This situation is common in cases where runoff is directed along the toe of a roadway embankment to a suitable crossing location, in which case the flow must enter the culvert barrel at an angle.

Figure 7.2: Barrel Skew Angle

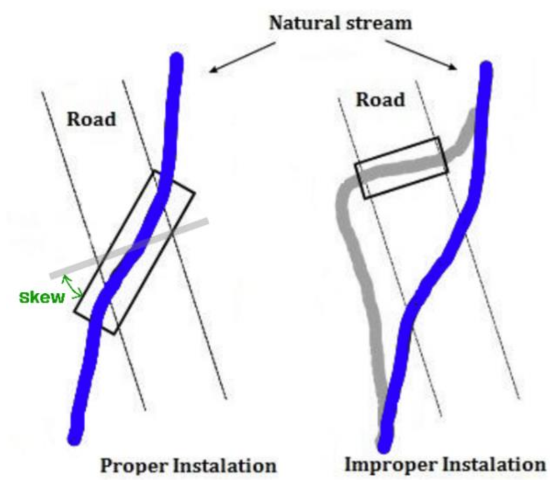


Figure 7.3: Barrel and Inlet Skew Angle
(USDOT, FHWA, HDS-5, 2012)

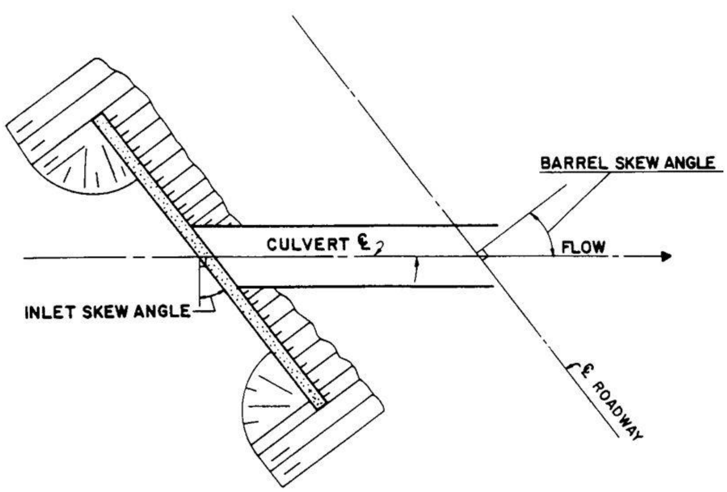
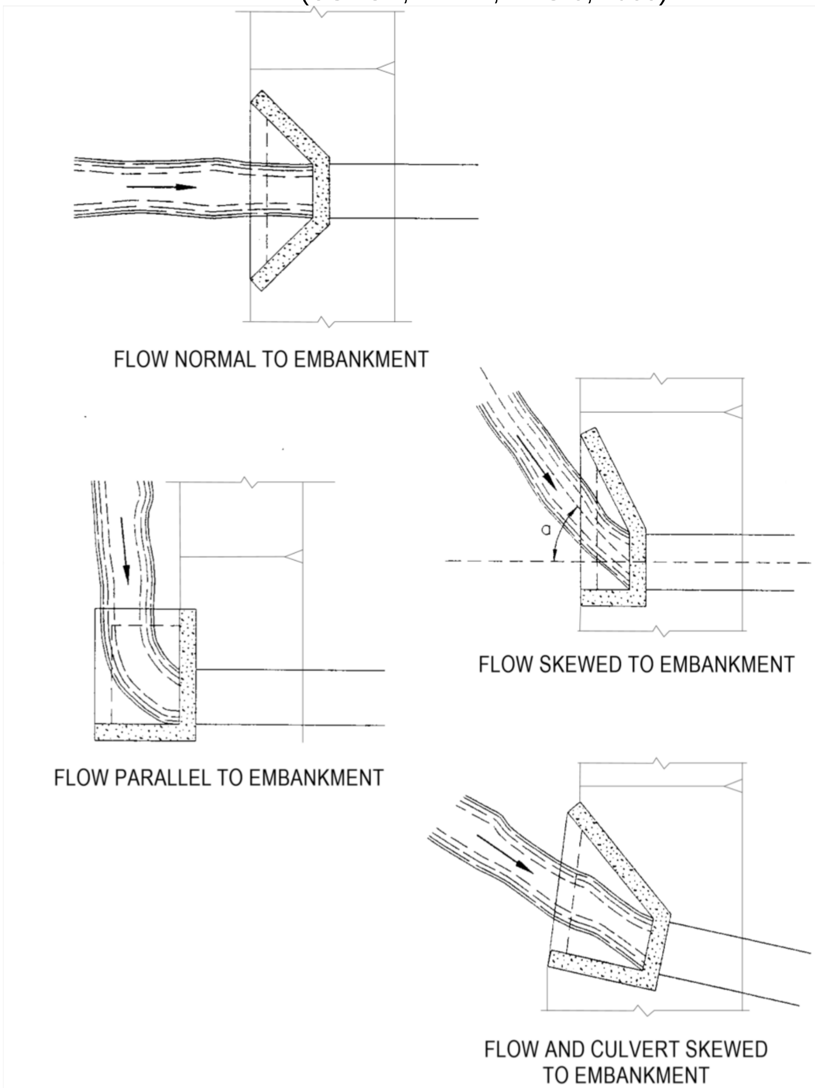


Figure 7.4: Typical Headwall/Wingwall Configurations for Skewed Channels

(USDOT, FHWA, HDS-5, 1985)

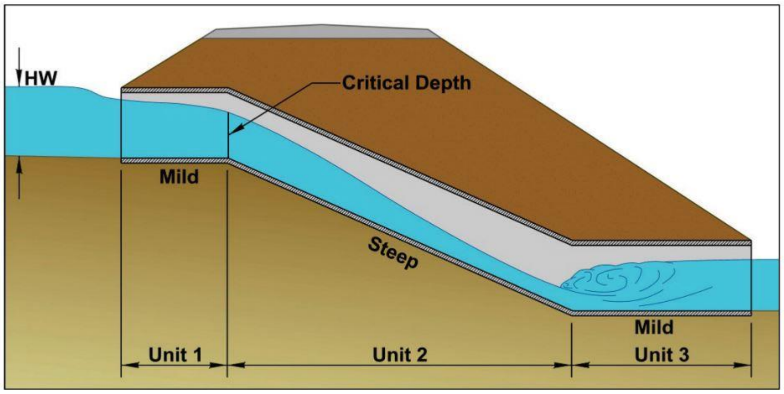


7.2.2.12 Bends and Slope Changes

A straight culvert alignment is desirable to reduce clogging risk and maintain hydraulic efficiency. However, site conditions may require a change of alignment, either horizontally or vertically. When considering a nonlinear culvert alignment, particular attention should be given to preventing sediment deposition or debris blockages within the culvert. Changes in culvert slope typically work well when they transition from a flatter to a steeper slope. However, care must be taken when transitioning from steeper to flatter slopes to ensure cleansing velocities are maintained over the full spectrum of flows throughout the reach downstream of the grade break. Cleansing velocities vary depending on the bed load and for clear water flows shall not be less than 3 ft/s, see Section 7.2.2.3. In addition to maintaining cleansing velocities, in all but clear water flow situations the vertical grade break should be located as close to the outlet as possible. Where distance is greater than 30 feet from the outlet, a manhole must be placed for maintenance access from above to the grade break. Lastly, easy access to the culvert outlet for maintenance equipment must be provided.

Figure 7.5: "Broken Back" Culvert

(USDOT, FHWA, HDS-5, 2012)



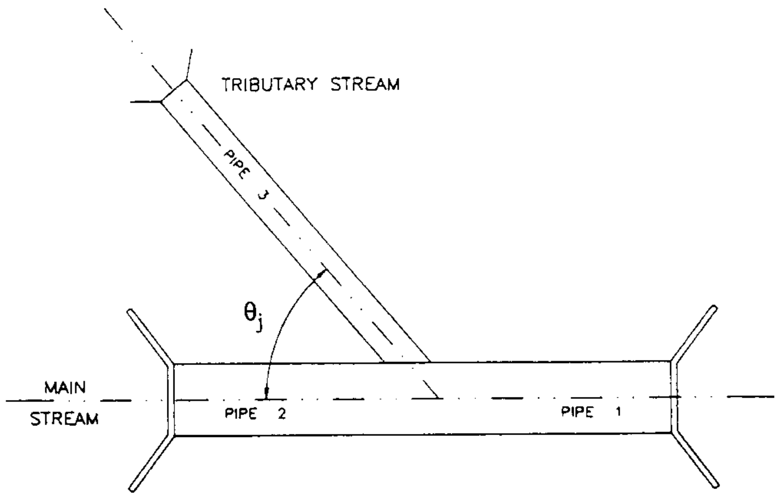
In designing culvert with horizontal curves, the energy losses due to the bends (aka bend losses) must be considered. If the culvert operates in inlet control, no increase in headwater occurs unless the bend losses cause the culvert to transition to outlet control. If the culvert operates in outlet control, an increase in energy losses and headwater will result due to the bend losses. See Chapter 4 for additional guidance on calculating losses.

The broken back culvert, shown in Figure 7.5, has four possible control sections: the inlet (due to the size and shape of the opening), the outlet (due to either a high tailwater or friction losses and bend within the culvert), and the two breaks in slope. The upstream vertical grade break may act as a control section, with the flow passing through critical depth just upstream of the grade break. In the case illustrated, Unit 1 of the culvert is flowing at subcritical depth and therefore operates under outlet (downstream) control. Unit 2 in the illustration is flowing at supercritical depth and, therefore, operates under inlet (upstream) control. Outlet control calculation procedures can be applied to Unit 1, assuming critical depth at the grade break, to obtain a headwater elevation. The controlling flow condition is that which produces the highest headwater elevation.

7.2.2.13 Junctions

Where possible flow from two or more systems should join downstream of a road culvert or through the downstream headwall of the culvert barrel. However, where this is not feasible, flow from two or more systems may be combined at a junction as illustrated below in Figure 7.6.

Figure 7.6: Culvert Junction
(USDOT, FHWA, HDS-5, 2012)



Headloss at the junction may be an important factor in the hydraulic design of a culvert containing a junction, particularly where the lateral flows are large. This is due to the loss in momentum resulting from the change in direction of the lateral flows, as well as from turbulence associated with the junction. Attention should be given to streamlining (minimizing the angle between the main line and the lateral) the junction to minimize turbulence and head loss. Also, timing of peak flows from the two branches should be considered in analyzing flow conditions and control.

Where culvert barrels are operating under inlet control, junction losses may have no bearing on the culvert design. However, in all cases outlet control must be checked.

7.2.2.14 Trash Racks and Access Barriers

For trash racks with approach velocities less than 3 feet per second, it is not necessary to include a head loss for the trash rack; however, for velocities greater than 3 feet per second, such computations are required. See Hydraulic Structures, Chapter 9 , Section 9.7 for further discussion regarding trash racks.

7.2.2.15 Flotation and Anchorage

Flotation is the term used to describe the failure of a culvert due to the uplift forces caused by buoyancy. The buoyant force is produced from a combination of high head on the outside of the inlet, saturation of the embankment fill soils and the large region of low pressure on the inside of the inlet caused by flow separation. As a result, a large bending moment is exerted on the end of the culvert. If there is no water in the soil outside of the pipe, then there are no buoyant forces.

Where the pipe projects from an embankment slope, the risk of failure is extremely high as the pipe is surrounded by water and the buoyant force is great due to flow separation in the pipe. In a typical unlined embankment where compaction directly over the pipe at the slope face is difficult, the soils are easily saturated resulting in large buoyant forces. If the slope is concrete lined, the embankment material below the lining is dry and there are no buoyant forces.

Flotation can also be caused by flows that overtop the roadway and damage or remove embankment materials on top of the culvert. This can be mitigated by constructing areas of potential overtopping with concrete or other lining materials that can withstand the overtopping forces.

Debris blocking the culvert end or damage to the inlet can also increase the risk of flotation. The resulting uplift may cause the inlet ends of the barrel to rise and bend. Occasionally, the uplift force is great enough to dislodge the embankment. Generally, flexible culvert materials are more vulnerable to failure of this type because of their light weight and lack of resistance to longitudinal bending. Large, projecting, or mitered corrugated metal culverts are the most susceptible.

See also Section 6.2.11 for buoyancy mitigation.

Several additional precautions can be taken by the designer to guard against flotation of a short road culvert. Ensure steep slopes (1 to 1 or steeper) along the embankment fill over the culvert (when fill is more than 1.5 times the pipe diameter) are protected against erosion by slope paving or headwalls. This will also help inlet and outlet stability. When embankment fill heights are less than 1.5 times the pipe diameter or fill slopes are flatter than 1 to 1, the designer may consider other applications such as concrete encasement, concrete headwalls, and anchored-down tie bars to guard against failures caused by flotation. Limiting headwater buildup also helps prevent flotation. Where headwater depths are greater than 1.5 times the culvert height, the designer should consider implementing mitigation measures to reduce the risk of damage to the culvert and embankment due to flotation.

7.2.2.16 Safety

Culverts shall be designed to conform to the safety protocols identified in Section 1.3.5. of this manual.

7.2.2.17 Inlets

Culvert inlets are used to transition the flow from upstream into the culvert barrel. Inlet losses have been studied extensively for several types of inlets. The inlet control nomographs included in Section 7.4.3 indicate the required headwater depth to pass the design discharge through the entrance to various culvert types (i.e. concrete pipe, corrugated metal pipe, box) and geometric configurations considering both the inlet shape (e.g. square edge, beveled edge, etc.) and approach geometry (e.g. straight headwall, flared headwall, etc.). The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the upstream approach channel is usually wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition, for example, utilizing a beveled vs. straight edge on a concrete pipe inlet, will lessen the energy loss and create a more hydraulically efficient inlet condition. Additional nomographs for various culvert types and configurations are contained in *Hydraulic Design of Highway Culverts* (USDOT, FHWA, HDS No. 5, April 2012).

It should be noted that improving culvert inlets will cause the greatest increase in culvert capacity when the culvert is operating under an inlet control condition. The hydraulic performance of culverts operating under inlet control can be improved by changing the geometry of the headwall at the inlet opening as well as the shape of the headwall itself (i.e., straight headwall vs. wingwall). The advantage of these improvements is to convert an inlet control culvert closer to outlet control by using more of the barrel capacity.

Inlet opening geometries, such as beveled-edge, side tapered, and slope-tapered (Figure 7.7), offer advantages over traditional square edge openings. For instance, a beveled-edge opening can increase the culvert capacity by as much as 20 percent over a square edge opening.

The District typically accepts Caltrans standards for pipe headwalls. For reinforced concrete pipes (RCP), see Caltrans Standard D89 and D90. For reinforced concrete boxes (RCB), see Caltrans Standard D82 – D84. Hydraulic analysis should represent the inlet edge condition per the standard plan.

Beveled edges at the entrance of the culvert, as shown in Figure 7.7, are a method of increasing inlet performance. Beveled edges reduce the contraction of flow by effectively enlarging the face of the culvert. The 1.5:1 (33.7 degrees) bevels require some structural modification but will provide slightly better inlet performance than the 1:1 (45 degree) bevels. The 1:1 bevels require very minor structural modifications of the culvert headwall and increase both inlet and outlet control performances. Therefore, the use of 1:1 bevels is recommended whether the culvert is in inlet or outlet control.

Side-tapered inlets have an enlarged face area accomplished by tapering box culvert sidewalls as shown in Figure 7.8. It provides an increase in flow capacity of 25 to 40 percent over square-edged inlet openings. There are two types of control sections for side-tapered inlets, which are face and throat control. The advantages of side-tapered inlets under throat control are reduced flow contraction angle at the throat due to the tapered side walls between the wingwall and regular box section, and increased head on the regular box section since the control occurs further downstream at the throat/box interface versus at the wingwall/box interface as would occur where a side-tapered inlet is not used.

Slope-tapered inlets provide additional head at the throat section as shown in Figure 7.9. This type of inlet can have over 100 percent greater capacity than a conventional culvert with a square-edge opening. The degree of increased capacity depends upon the drop between the face and the throat section. Both the face and the throat are possible control sections. The inlet face should be designed with a greater capacity than the throat to promote flow control at the throat and, therefore, greater potential capacity of the culvert. This type of inlet may not be appropriate for flows containing high sediment loads due to the transition from steeper to flatter slope within the culvert; caution should be excised for this design condition.

Figure 7.7: Inlet Bevel Detail

(USDOT, FHWA, HDS-5, 2012)

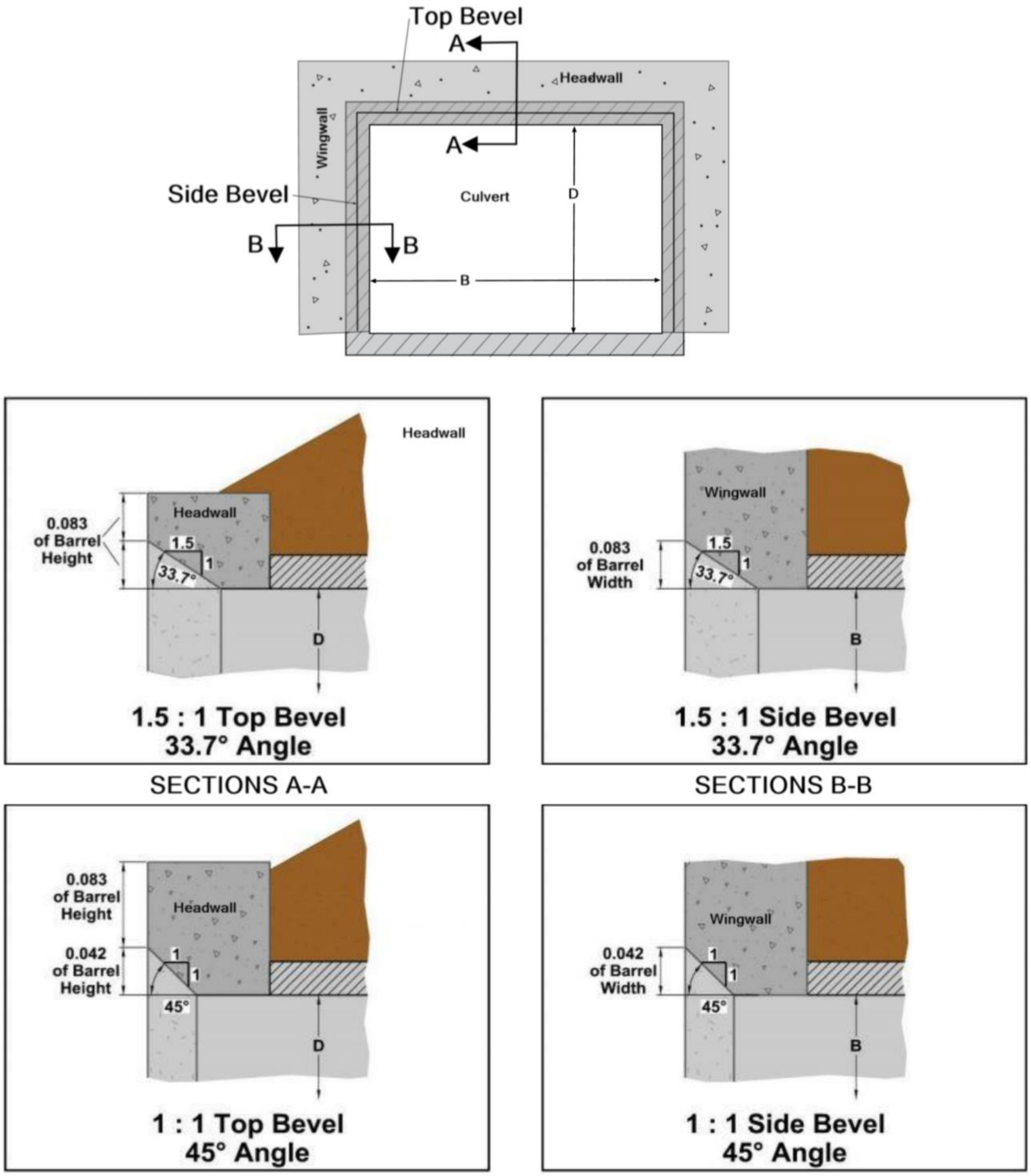


Figure 7.8: Side-Tapered Inlet
(USDOT, FHWA, HDS-5, 2012)

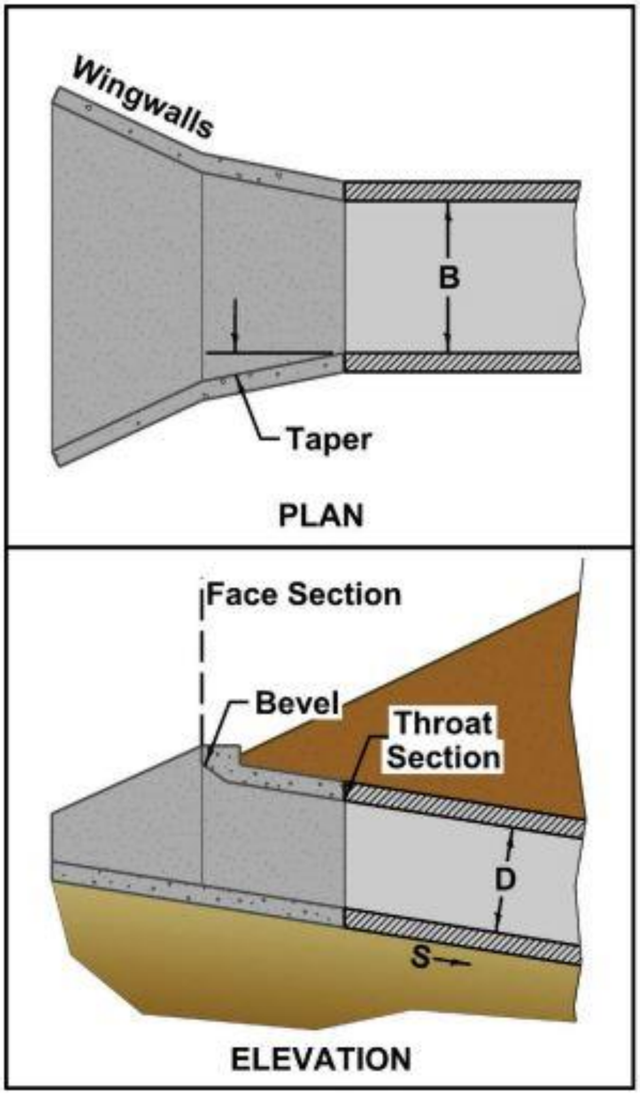
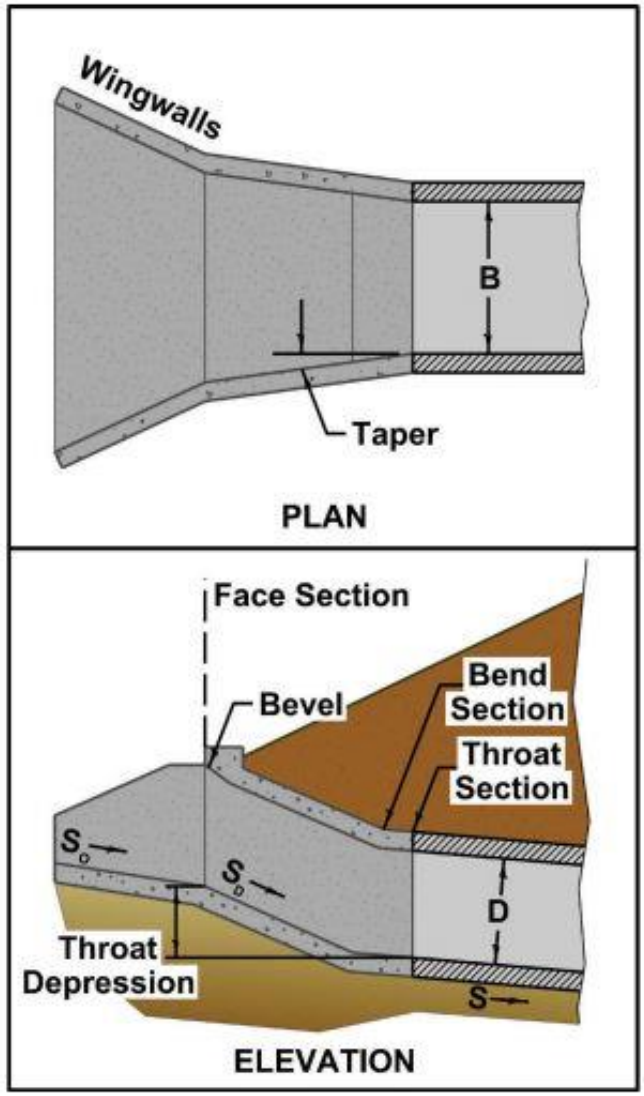


Figure 7.9: Slope-Tapered Inlet
(USDOT, FHWA, HDS-5, 2012)



7.2.2.18 Outlets

A culvert outlet likely represents a concentration of flow relative to what would have occurred naturally. To mitigate the risk of impacting the downstream system, the outlet must be carefully designed to 1) transition flow back to the natural conditions that would have existed in the natural condition (i.e., before the addition of the roadway embankment or culvert), and 2) prevent erosion or scour from damaging the culvert or roadway embankment.

Projecting culvert outlets are not permitted unless approved by the governing agency. The minimum requirement is to provide a preformed end section or headwall (with or without a wingwall configuration) with a cutoff wall provided at the end of the apron. Culvert outlet designs are presented in Section 7.3. Energy dissipation structures, if needed, are presented in Chapter 9 , Hydraulic Structures, Section 9.3.

7.3 ENTRANCES AND OUTLETS FOR CULVERTS

This Section provides guidelines for design of culvert type inlets and outlets to closed conduit systems. Runoff entering and exiting closed conduits may require transitions into and out of the conduit to minimize entrance losses and protect adjacent property and drainage facilities from possible erosion. Pavement drainage inlets that allow runoff to drop into catch basins are discussed in Chapter 5 , Section 5.2 and are not addressed here.

7.3.1 Interaction with Other Systems

Closed conduit inlets and outlets are the locations where the conduit transitions back to a natural or channelized condition. Erosion protection measures such as riprap or concrete energy dissipators may be required in the vicinity of the inlet and outlet to complete the transition to the design velocity and flow depth of the upstream/downstream channel. The design of inlets and outlets should consider all conditions in the upstream and downstream direction to the location where the inlet, outlet, and closed conduit have no effect on pre-design flow conditions.

7.3.2 Roadway Embankment Protection / Overtopping

Roadway embankments with culverts passing through them should be protected from potential damage caused by roadway overtopping during a runoff event in excess of the culvert design capacity. When a planned flow over the road has damage potential, such as when the 100-year discharge causes flow over the roadway, the embankment for downstream sides may need to be protected by use of paving, grouted riprap, or other means of permanent stabilization.

7.3.3 Entrance Structures and Transitions

Criteria for culvert entrances are contained in Section 7.2.2. Design considerations include aligning the culvert with the natural channel profile, protection against inlet failure due to buoyant forces, and safety considerations for the public.

Culvert performance can be improved by providing a smooth and gradual transition at the entrance. Improved inlet designs have been developed for culverts operating in inlet control and are presented in Section 7.2.2.

Supercritical flow transitions at inlets require special design consideration. For design of supercritical flow conditions, refer to *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, FHWA, HEC-14, 2006).

7.3.4 Outlet Velocity and Protection

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, toe down extensions or transitions shall be considered.

Standard measures for scour protection at conduit outlets include cutoff walls, wingwalls with aprons, and grouted or ungrouted riprap. These measures should be used as appropriate such that the velocity entering the receiving channel is within the allowable range of velocities for the receiving channel's surface material. Use the procedures in Chapter 9 , Section 9.3 for designing culvert outlet protection.

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 maybe determined by diving the discharge rate by the cross-section flow area within the barrel, based on the chosen depth.

7.3.5 Safety

Inlets and outlets to closed conduits may present dangers to the public when access is not controlled. Refer to Chapter 1 , Section 1.3.5 for the safety requirements related to conduit inlets and outlets.

7.4 DESIGN PROCEDURES

7.4.1 Hydraulic Control Scenarios

Culverts function either in inlet control or outlet control depending on the flow rate entering and being conveyed through the system. *At minimum*, the following scenarios must be analyzed and the scenario providing the most limiting condition is used for design:

- **Inlet Control:** In inlet control scenarios, the amount of water that can flow through the culvert is being limited by the geometry at the inlet. The following scenarios must be checked:
  - **Critical Depth Control:** Critical control typically occurs when the depth of water upstream of the culvert is relatively low relative to the internal height of the conduit. In this scenario, water will drop through critical depth (of the conduit) as it enters the culvert. This condition is analyzed using the following formula:

$H = D_c + (Losses)$, where

- H Is the depth of water immediately upstream of the inlet, ft
- Dc is for the culvert barrel is calculated per Section 4.1.2.4, using the desired flow rate, ft
- Losses include minor losses at the inlet per Section 4.3.4.4 and losses due to any trash/debris grate per Section 9.7, using the desired flow rate

  - **Orifice Control:** Orifice control occurs after the inlet has been fully submerged by the headwater, but flow within the culvert is still flowing 'open'. This condition is analyzed following the methods described in Section 9.4.

- **Outlet Control**

  - **Friction Control**: Friction control occurs when the size, slope, and material of the culvert barrel(s) are limiting the amount of flow that can enter the culvert. In these scenarios the culvert barrels are flowing full (subcritical), and the capacity and upstream depth is determined using the methods described in Section 4.4 beginning at an appropriate downstream control point.

7.4.2 Preferred Analysis Methods

The hydraulic design of culverts can be performed by any of the following methods:

- Design of the culvert as a 'storm drain' following the methods covered in Chapter 4 of this manual. This method is required if the culvert system is long (similar to a storm drain) or involves complex grade breaks, junctions, etc.
  - The water surface profile calculations must extend a reasonable distance upstream of the culvert inlet to account for the conditions in the approach channel/basin (cross section, depth, velocity), and account for losses as water enters the culvert, such as: Transition Losses, Inlet Losses, and other Losses such as related to any debris barriers.
  - When determining the depth of water upstream of the inlet, check the inlet control scenarios described in Section 7.4.1 and compare them to depth determined for the the upstream system determined from the methods in Chapter 4 and use the highest upstream depth calculated.
- Following the methods described in Chapter 3 of USDOT, FHWA, HDS-5, 2012.
  - Section 3.5 of the HDS-5 describes culvert computations using software. Software acceptable to the District is included in the approved software list (rcflood.org>Business>Engineering Tools>Hydrology & Hydraulics>District Approved Software)
  - Nomographs in accordance with HDS-5 and provided in Section 7.4.2 and 7.4.3

7.4.3 Nomograph Design Method

This design method provides a convenient and organized procedure for designing culverts, considering inlet, and outlet control.

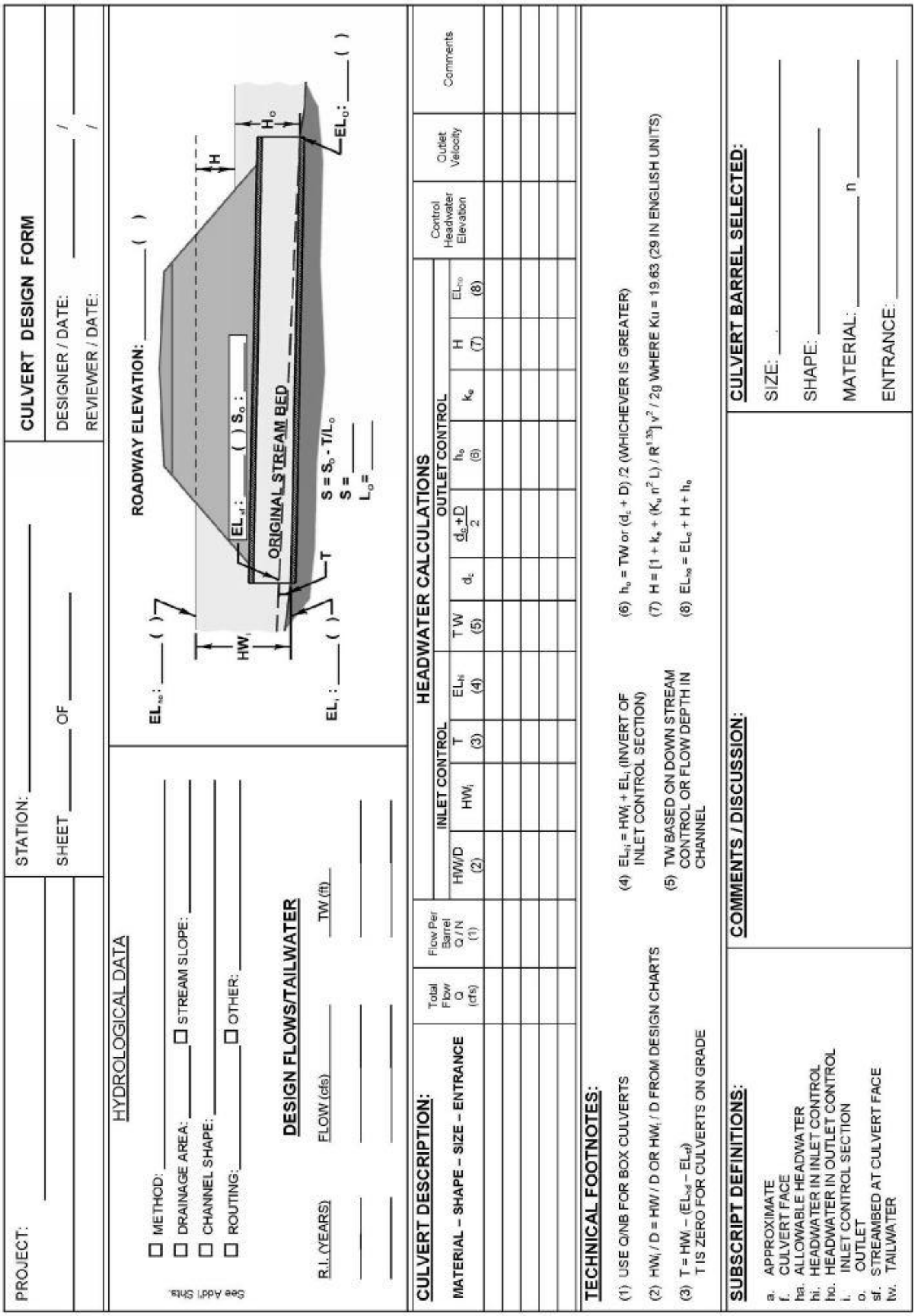
The first step in the design process is to summarize all known data for the culvert at the top of the

FHWA Culvert Design Form (Figure 7.10). This includes establishing a maximum allowable design headwater elevation, considering: roadway overflow elevation, finished floor elevation of structures within the backwater limits, and right of way or easement requirements for the backwater ponding elevation. This information is to be collected or calculated prior to performing the culvert design. The next step is to select a preliminary culvert material, shape, size, and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations.

The nomographs presented in Section 7.4.3 are a subset of the nomographs available in USDOT, FHWA, HDS-5, 2012. The nomographs included in this document are the culvert types more typically used in Riverside County. In lieu of nomographs, computational approaches (e.g. WSPGW, etc.) per the approved software list (rcflood.org>Business>Engineering Tools>Hydrology & Hydraulics>District Approved Software) may be used.

Figure 7.10: Culvert Design Form

(USDOT, FHWA, HDS-5, 2012)



PROJECT: ______

STATION: ______

SHEET ______ OF ______

CULVERT DESIGN FORM

DESIGNER / DATE: ______ / ______

REVIEWER / DATE: ______ / ______

HYDROLOGICAL DATA

See Add'l Shts.

☐ METHOD: ______

☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______

☐ CHANNEL SHAPE: ______

☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

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

CULVERT DESCRIPTION:

MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	HEADWATER CALCULATIONS – INLET CONTROL: HW/D (2)	HW_i	T (3)	EL_{hi} (4)	OUTLET CONTROL: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	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 Ku = 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: ______

7.4.3.1 Inlet Control

Inlet control depends on the cross-sectional area of the barrel, inlet geometry (square, round, etc.), inlet edge condition (beveled edge, square edge, groove end, etc.), headwall configuration (or lack of headwall), and the headwater (difference between the ponding elevation and upstream invert elevation). Inlet control occurs when the culvert barrel can convey more flow than the inlet will accept. In general, these kinds of culverts are steep (supercritical flow regime) and have a free-flowing outlet. The control section of a culvert operating under inlet control is located just inside the entrance. Shallow high-velocity flow within the barrel(s) characterizes culverts under inlet control. Because inlet hydraulic control is upstream, only the headwater and the inlet characteristics as described above affect culvert performance.

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration if the culvert is operating in inlet control. The inlet control nomographs in Section 7.4.3 are used in the design process. For the following discussion, refer to the schematic inlet control nomograph shown in Figure 7.11.

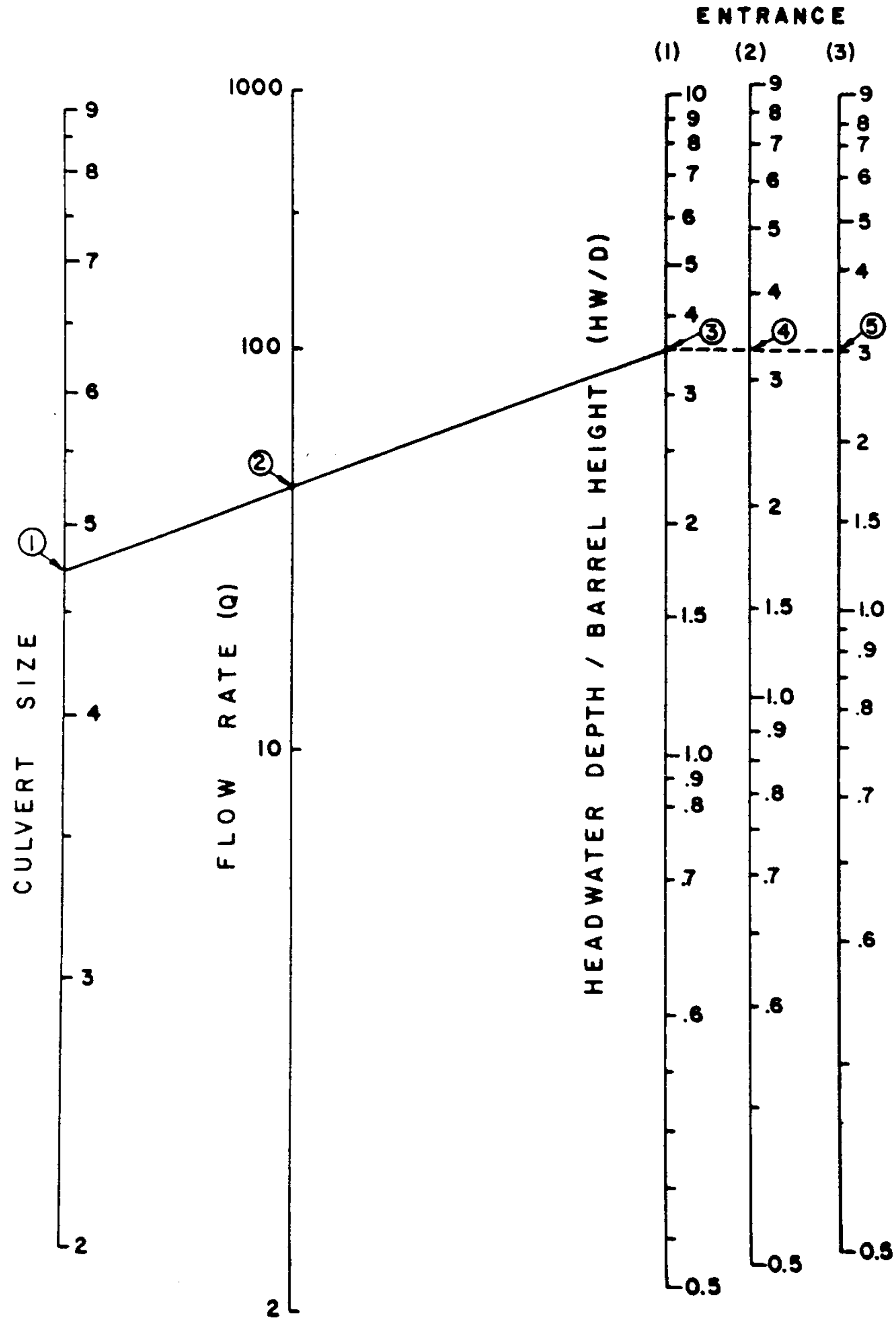
1. Locate the selected culvert size (point 1) and flow rate (point 2) on the appropriate scales of the inlet control nomograph. (Note that for box culverts, the flow rate per foot of barrel width is used.)
2. Using a straightedge, extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (HW/D) scale (point 3). The first HW/D scale is also a turning line.
3. If another HW/D scale is required, extend a horizontal line from the first HW/D scale (the turning line) to the desired scale and read the result.
4. Multiply HW/D by the culvert height, D, to obtain the required headwater (HW) from the invert of the control section to the energy grade line. HW equals the required headwater depth. If trashracks are used, add trashrack losses to HW.
5. Calculate the inlet control headwater elevation.

   $$EL_{hi} = EL_i + HW$$

   where EL_i is the invert elevation at the inlet.
6. If the inlet control headwater elevation exceeds the design headwater elevation determined in the first step and tabulated on Figure 7.10, a new culvert configuration must be selected, and the process repeated. Improvements to the inlet may suffice, or an enlarged barrel may be necessary, particularly if the outlet control headwater elevation calculated in the following Section also exceeds the design headwater elevation.

Figure 7.11: Inlet Control Nomograph (Example only, see Section 7.4.3)

(USDOT, FHWA, HDS-5, 2012)



7.4.3.2 Outlet Control

Outlet control involves the additional consideration of the tailwater in the outlet channel, and the slope, roughness, and length of barrel. 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 typically located at the barrel exit or further downstream. However, friction control is also considered outlet control in this worksheet (e.g., free outfall condition where the pipe slope is flat, and thus the control is not the outlet or downstream of the outlet but is instead the pipe itself).

The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert if the culvert is operating in outlet control. The nomograph to determine head for concrete pipe culvert flowing full, is used in the design process. See example in Figure 7.12.

1. Determine the tailwater depth above the outlet invert (TW) at the design flow rate. This is obtained from backwater or normal depth calculations of the downstream channel, or from field observations. Field observations are important in determining tailwater depths. The area downstream of the culvert should be examined for features that may create backwater effects, i.e., channel control, another culvert, etc. If such features are found, appropriate backwater analysis techniques should be employed to determine the tailwater depth. When culverts are in series, the headwater for one culvert can control the tailwater for the next upstream culvert and the culverts must be sized accordingly.
2. Calculate critical depth (d_c) based on the flowrate and culvert barrel height (D).
3. Calculate $(d_c + D)/2$.
4. Determine the depth from the culvert outlet invert to the hydraulic grade line (h_o).

   $h_o = TW$ or $(d_c + D)/2$, whichever is larger

5. From Table 7.1 (on page 7-30) obtain the appropriate entrance loss coefficient, K_e, for the culvert inlet configuration.
6. Determine the losses through the culvert barrel, H, using the nomograph to calculate head for concrete pipe culvert flowing full (Figure 7.12) or appropriate equations if outside the range of the nomograph.
   a) If the Manning's n-value given in the nomograph is different than the Manning's n for the culvert, adjust the culvert length using the equation:

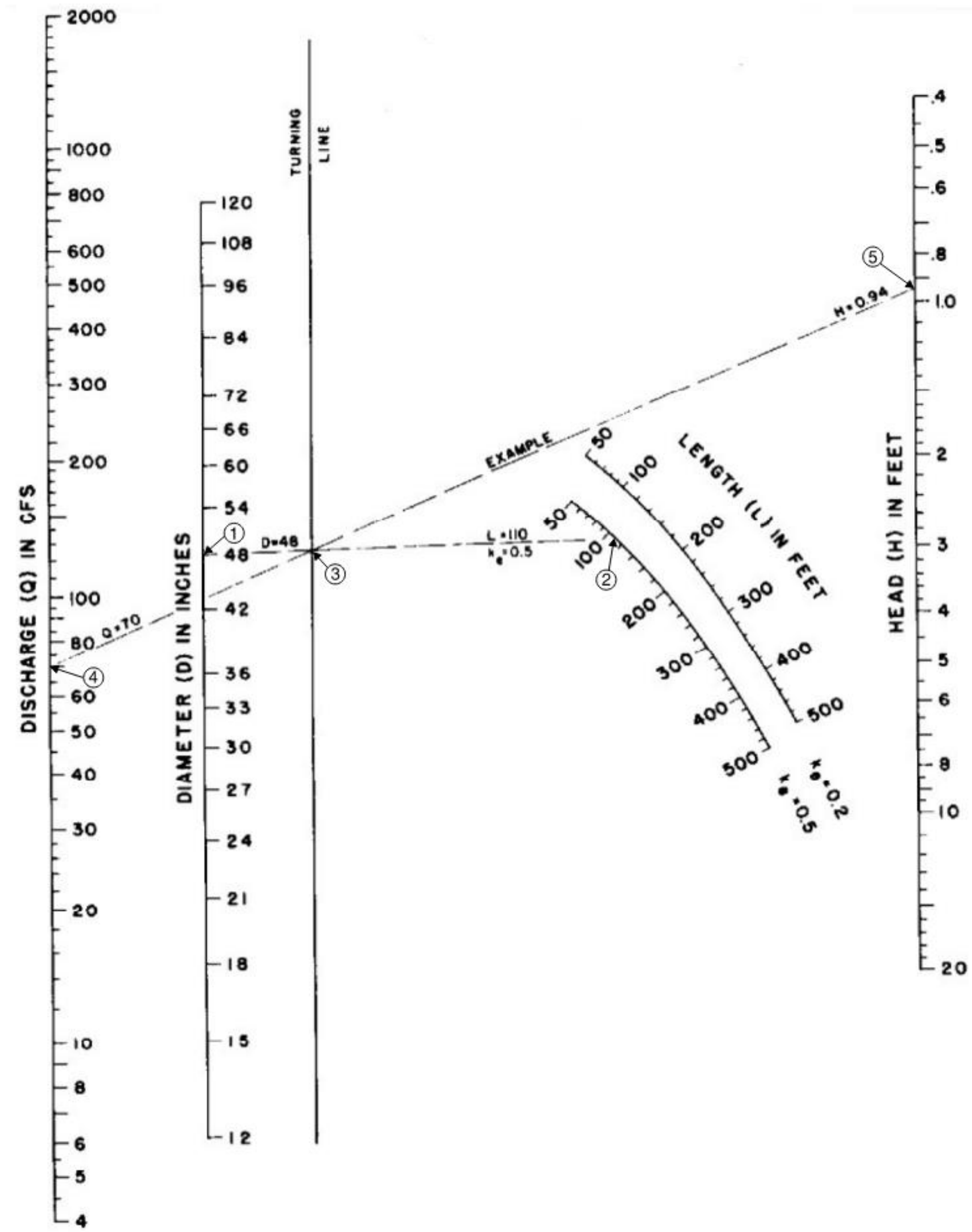
   $$L_1 = L\left(\frac{n_1}{n}\right)^2 \qquad (7.1)$$

   Then use L_1 rather than the actual culvert length when using the nomograph.

b) Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate K_e scale (point 2). This defines a point on the turning line (point 3).

Figure 7.12: Head for Concrete Pipe Culvert Flowing Full Nomograph (example only)

(USDOT, FHWA, HDS-5, Chart 5B, 2012)



c) Again, using the straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the Barrel Losses (H) scale. Read H, which is the energy loss through the culvert, including entrance, friction, and outlet losses.

d) All other applicable losses (e.g., bend and junction) should be added to H.

7. Calculate the headwater elevation.

$$EL_{ho} = EL_o + H + h_o \tag{7.2}$$

where EL_o is the invert elevation at the outlet.

8. If the outlet control headwater elevation exceeds the design headwater elevation determined in the first step and tabulated on Figure 7.10, a new culvert configuration may need to be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

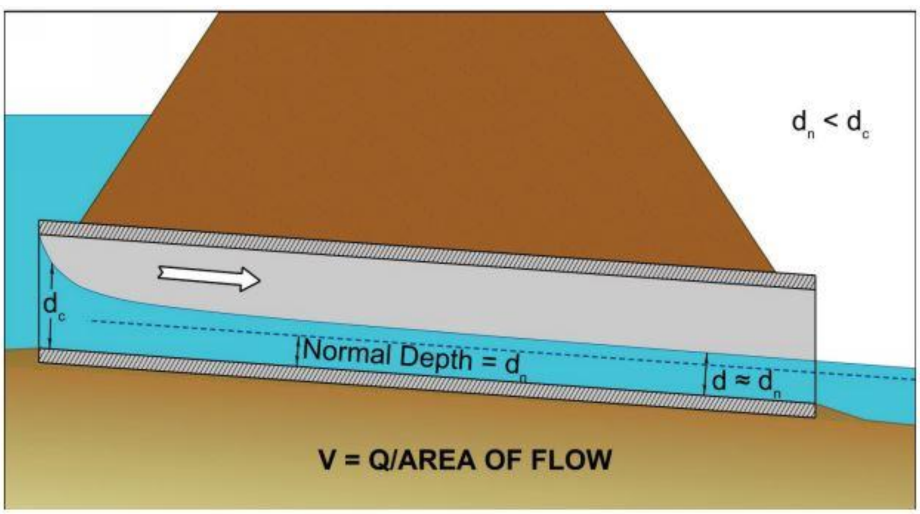
7.4.3.3 Evaluation of Results

Results of the analysis are determined by comparing the headwater elevations calculated for inlet and outlet control. The higher of the two is designated as the controlling headwater elevation. The culvert can be expected to operate with this controlling headwater during the design (peak) flow rate.

The outlet velocity is calculated as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity (Figure 7.13). Normal depth for circular and rectangular culverts can be found using Figure 7.18.

Figure 7.13: Outlet Velocity – Inlet Control
(USDOT, FHWA, HDS-5, 2012)

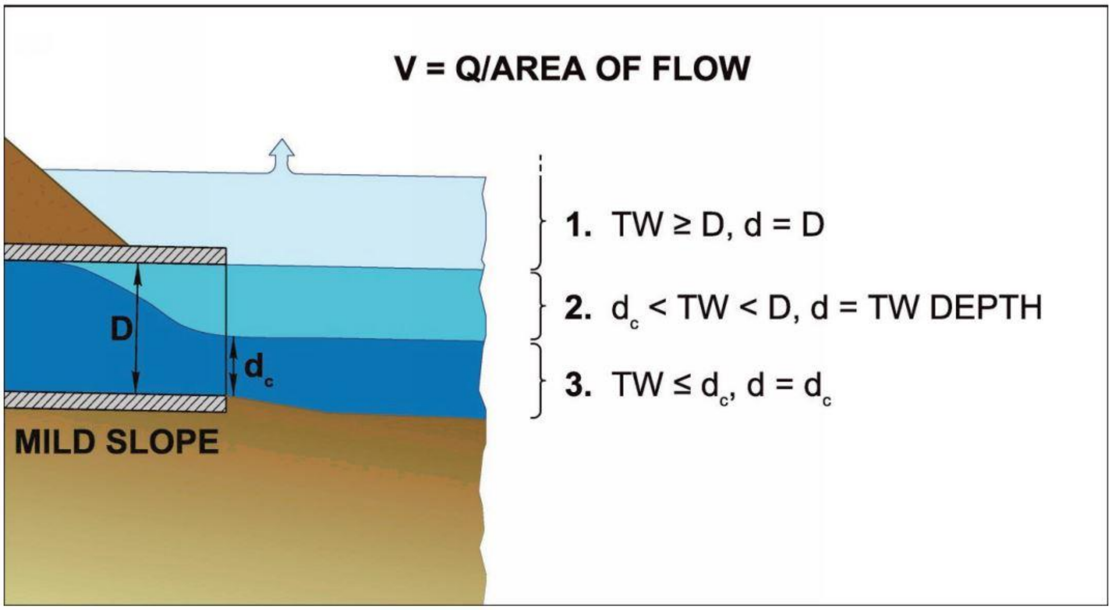


(Area based on Barrel geometry and depth equal to Normal Depth)

2. If the controlling headwater is in outlet control, determine the area of flow and velocity at the outlet based on the barrel geometry (see Figure 7.14) and the following:
   a) Critical depth, if the tailwater is below critical depth.
   b) The tailwater depth if the tailwater is between critical depth and the top of the barrel.
   c) The height of the barrel if the tailwater is above the top of the barrel.

Figure 7.14: Outlet Velocity - Outlet Control

(USDOT, FHWA, HDS-5, 2012)



Repeat the design process until an acceptable culvert configuration is determined. Once the barrel is selected it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, and the headwalls and wingwalls must be dimensioned.

If outlet control governs and the headwater depth (referenced to the inlet invert) is less than 1.2D, it is possible that the barrel flows partly full through its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or $(d_c + D)/2$. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below 0.75D, the approximate method should not be used.

If the selected culvert will not fit the site, return to the culvert design process, and select another culvert. After a selected culvert is found to meet the design conditions, document the design to this point. Culvert design documentation may include a performance curve which displays culvert behavior over a range of discharges. Development of performance curves is presented later in this Section, and Example 3 in Section 7.4.4 contains a performance curve calculation.

Additional design considerations including stage discharge ratings, roadway overtopping, and performance curves, are discussed in the following sections.

7.4.3.4 Stage Discharge Ratings

Culverts are frequently used for detention basin outlet structures. The culvert design methods presented in this Section can be used to develop the stage-discharge relationship for routing flows through these structures. If the detention basin discharges into a storm drain system, procedures from Section 6.2 should be used to establish the hydraulic grade line for that storm drain to check for outlet control.

All reservoir routing procedures require three basic data inputs: 1) an inflow hydrograph; 2) a stage versus storage relationship; and 3) a stage versus discharge relationship. Stage, that is the elevation above some base datum, is the parameter which relates storage to discharge providing the key to the storage routing solution.

Stage versus discharge data can be computed from culvert data and the roadway geometry as described below under Performance Curves. Discharge values for the selected culvert and over-topping flows are tabulated with reference to elevation. The combined discharge is utilized in the formulation of a performance curve.

7.4.3.5 Performance Curves

Performance curves are representations of flow rate versus headwater depth or stage for a culvert. Because a culvert has several possible control sections (inlet, outlet, throat), a given installation will have a performance curve for each control section and one for roadway overtopping. The overall culvert performance curve is made up of the controlling portions of the individual performance curves for each control section.

Inlet Control - Inlet control performance curves may be developed using the inlet control nomographs provided in Section 7.4.3. The headwater depths corresponding to the series of flow rates are determined and then plotted. The transition zone is inherent in the nomographs.

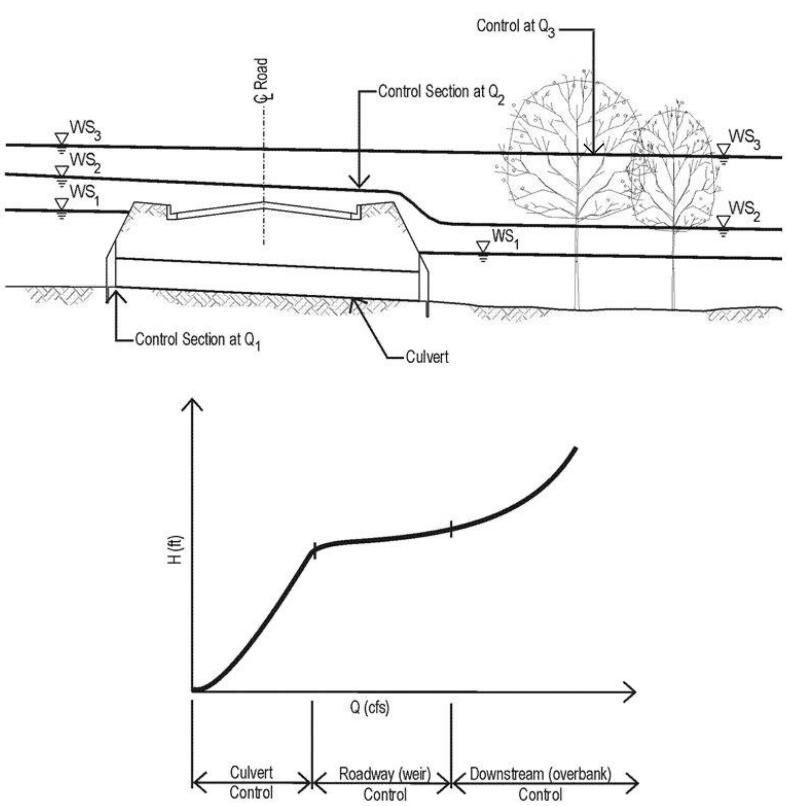
Outlet Control - Outlet control performance curves may be developed using the outlet control nomographs provided in Section 7.4.3. Flows bracketing the design flow are selected. For these flows, the total losses through the barrel are calculated or read from the outlet control nomographs. The losses are added to the elevation of the hydraulic grade line at the culvert outlet to obtain the headwater.

If backwater calculations are performed beginning at the downstream end of the culvert, friction losses are accounted for in the calculations. Adding the inlet loss to the energy grade line in the barrel at the inlet results in the headwater elevation for each flow rate. An example of development of a performance curve is presented in Example 3 in Section 7.4.4.

Roadway Overtopping - A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial-and-error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed. The performance curve depicts the sum of the flow through the culvert and the flow across the roadway.

Figure 7.15: Culvert Performance Curve with Roadway Overtopping

(USDOT, FHWA, HDS-5, 1985)



The overall performance curve can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated. It is recommended that the 2-, 10-, 50-, and 100-year flow rates be included in the range of flow rates considered.
2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert based on the controlling stage for each discharge.
3. When the culvert headwater stages exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation (7.6) to calculate flow rates across the roadway.
4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Using the combined culvert performance curve, it is an easy matter to determine the headwater stage for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates. When roadway overtopping begins, the rate of headwater increase will diminish. The headwater will rise very slowly from that point on. Figure 7.15 depicts an overall culvert performance curve with roadway overtopping. Example 3 in Section 7.4.4 illustrates the development of an overall culvert performance curve. The 100-year discharge should be identified on the performance curve and the corresponding water surface elevation.

The Federal Highway Administration's computer program, HY-8 (USDOT, 1999), can be used in the development of performance curves. HY-8 automates the design methods described in HDS-5 (USDOT, 2012) and HEC-14 (USDOT, 2006). The U.S. Army Corps of Engineers HEC-RAS computer programs (USACE, 2001a, and 2001b) are also capable of analyzing culverts. For simplicity, the use of HY-8 is preferred for design of culverts that are not subject to backwater conditions. HEC-RAS requires additional expertise to use proficiently and may be more appropriate for complex analyses such as design of culverts in river systems where downstream backwater effects impact culvert performance.

Roadway overtopping will begin as the headwater rises to the elevation of the lowest point of the roadway. This type of flow is similar to flow over a broad-crested weir. The length of the weir can be taken as the horizontal length along the roadway. The flow across the roadway is calculated from the broad-crested weir equation:

$$Q_o = C_d L_x (HW_r)^{1.5} \tag{7.3}$$

where:

- Q_o = overtopping flow rate, cfs (m^3/s)
- C_d = overtopping discharge coefficient = $K_t C_r$ from Figure 7.16 [C_d (SI) = 0.552 (C_d from Figure 7.16)]
- L_X = length of roadway crest, ft (m)
- HW_r = upstream depth, measured from roadway crest to water surface upstream of weir drawdown, ft (m)

The charts in Figure 7.16 provide estimates of the correction factors K_t *and* C_r. Chart A is for deep overtopping. Chart B is for shallow overtopping. Chart C is a correction factor for downstream submergence. Submergence occurs as the tailwater begins to encroach on the fee overfall from the weir.

If the elevation of the roadway crest varies, for instance where the crest is defined by a roadway sag vertical curve, the vertical curve can be approximated as a series of horizontal segments. The flow over each is calculated separately and the total flow across the roadway is the sum of the incremental flows for each segment (Figure 7.17).

The total flow across the roadway then equals the sum of the roadway overflow plus the culvert flow. A performance curve must be plotted including both culvert flow and road overflow. The headwater depth for a specific discharge, such as the 100-year discharge can then be read from the curve. Design Example 3 in Section 7.4.4 illustrates this procedure.

Figure 7.16: Discharge Coefficient and Submergence Factor for Roadway Overtopping
(USDOT, FWHA, HDS-5, 2012)

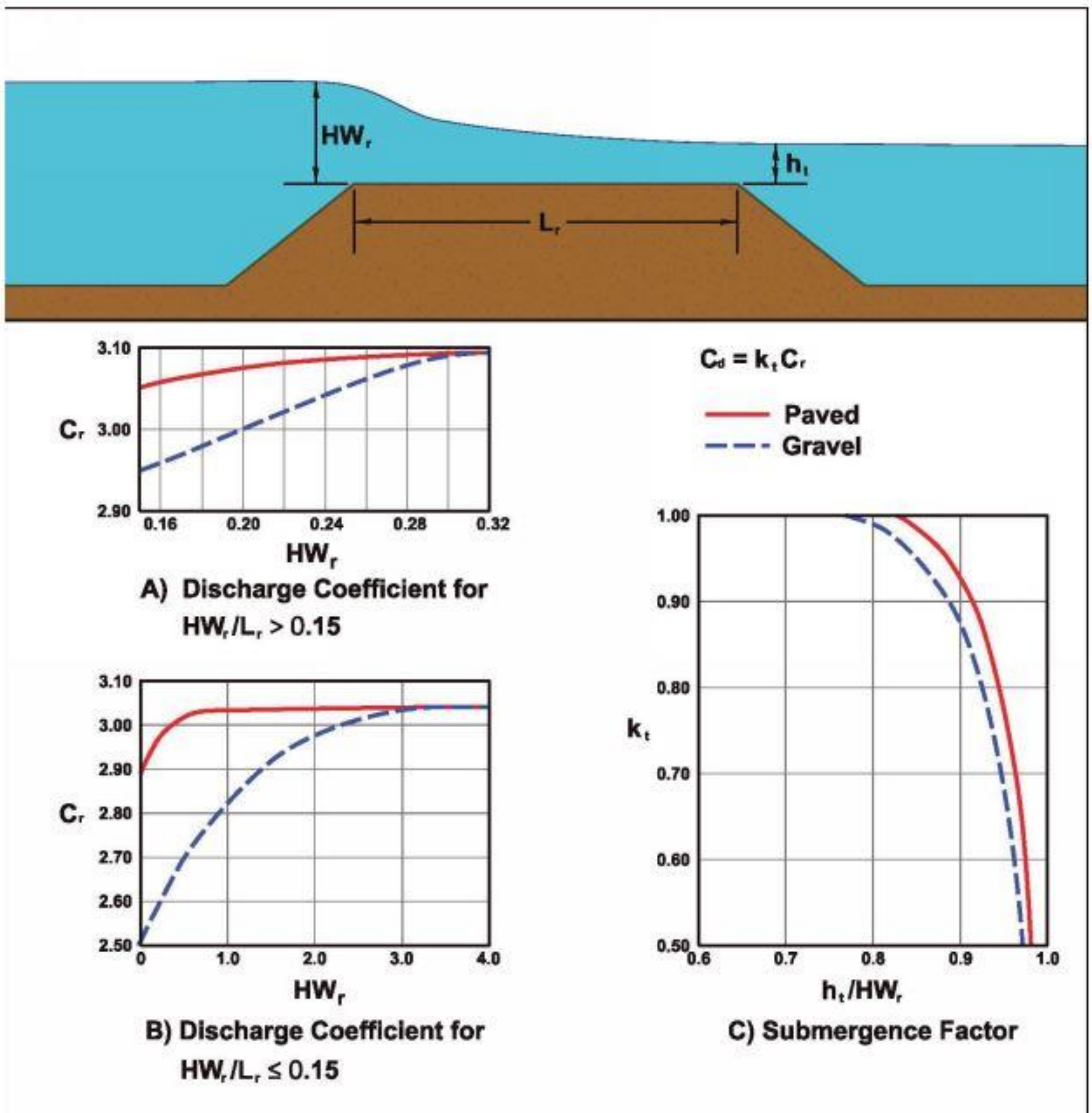
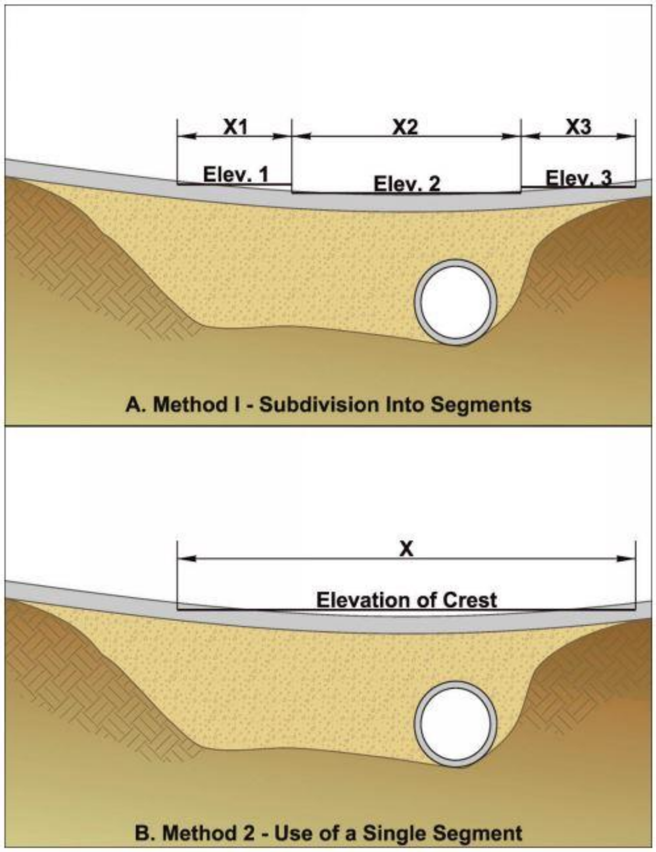


Figure 7.17: Weir Crest Length Determinations for Roadway Overtopping
(USDOT, FHWA, HDS-5, 2012)



7.4.4 Design Aids

Computer programs for culvert design are acceptable provided they are based on USDOT, FHWA, HDS-5, 2012 and listed on the approved software list (https://rcflood.org/Business/Engineering-Tools).

The Culvert Design Form (Figure 7.10) has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data are also included. At the top right is a small sketch of a culvert with blanks for inserting important dimensions and elevations.

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet control and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert barrel selected. The design chart should be completely filled out, including consideration of inlet and outlet control. Table 7.1 and Figure 7.18 through Figure 7.29 are provided to facilitate completion of the Culvert Design Form.

Table 7.1: Entrance Loss Coefficients

OUTLET CONTROL, FULL OR PARTLY FULL ENTRANCE HEAD LOSS
(USDOT, FHWA, HDS-5, 2012)

Type of Structure and Design of Entrance	Coefficient, K_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (grove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45^o bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45^o bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled on sides	0.2
Wingwalls at 30^o to 75^o to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwalls at 10^o to 25^o to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Note: WSPGW assumes a square edge.

Figure 7.18: Curves for Determining the Normal Depth

(Chow, 1959)

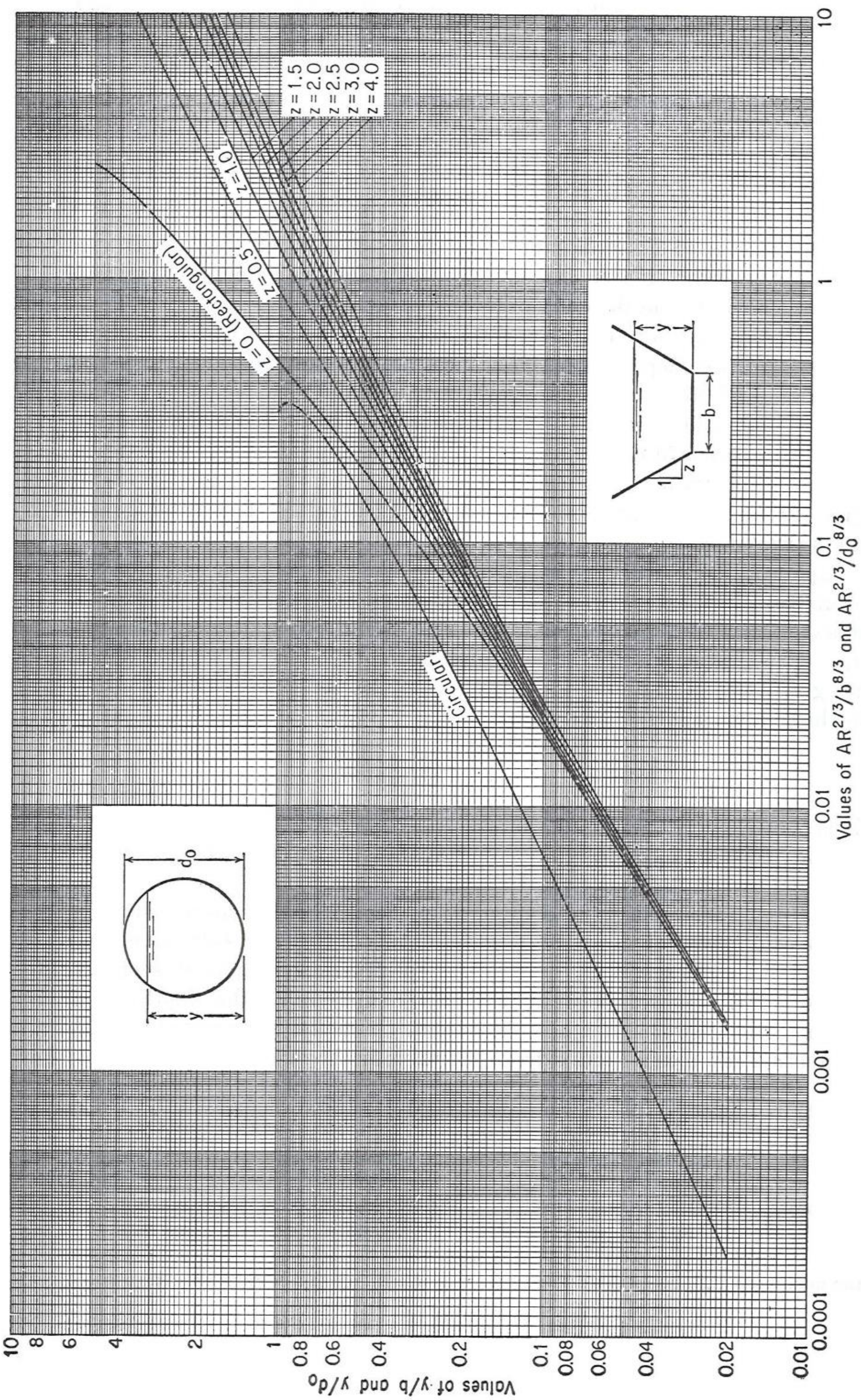
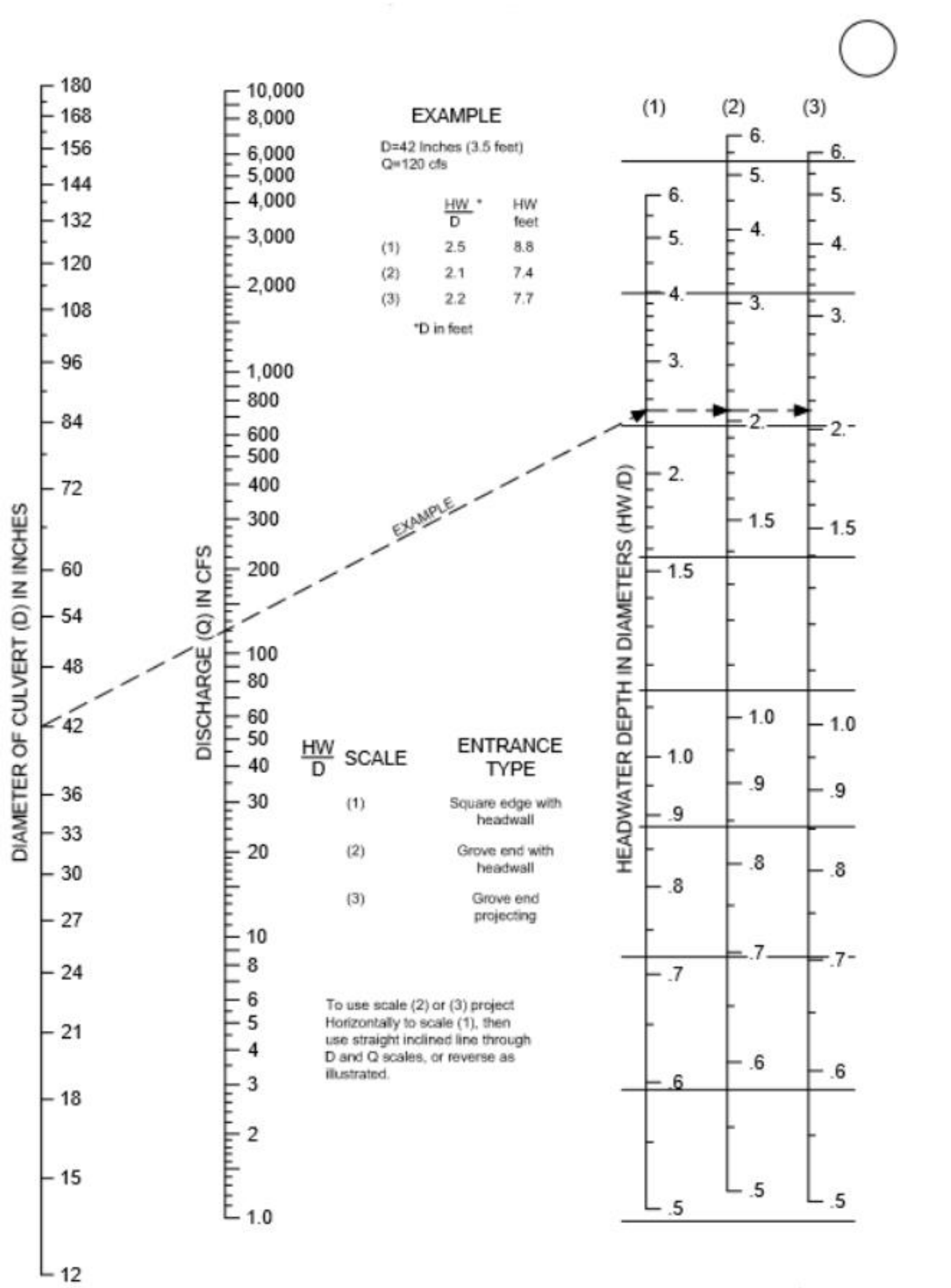


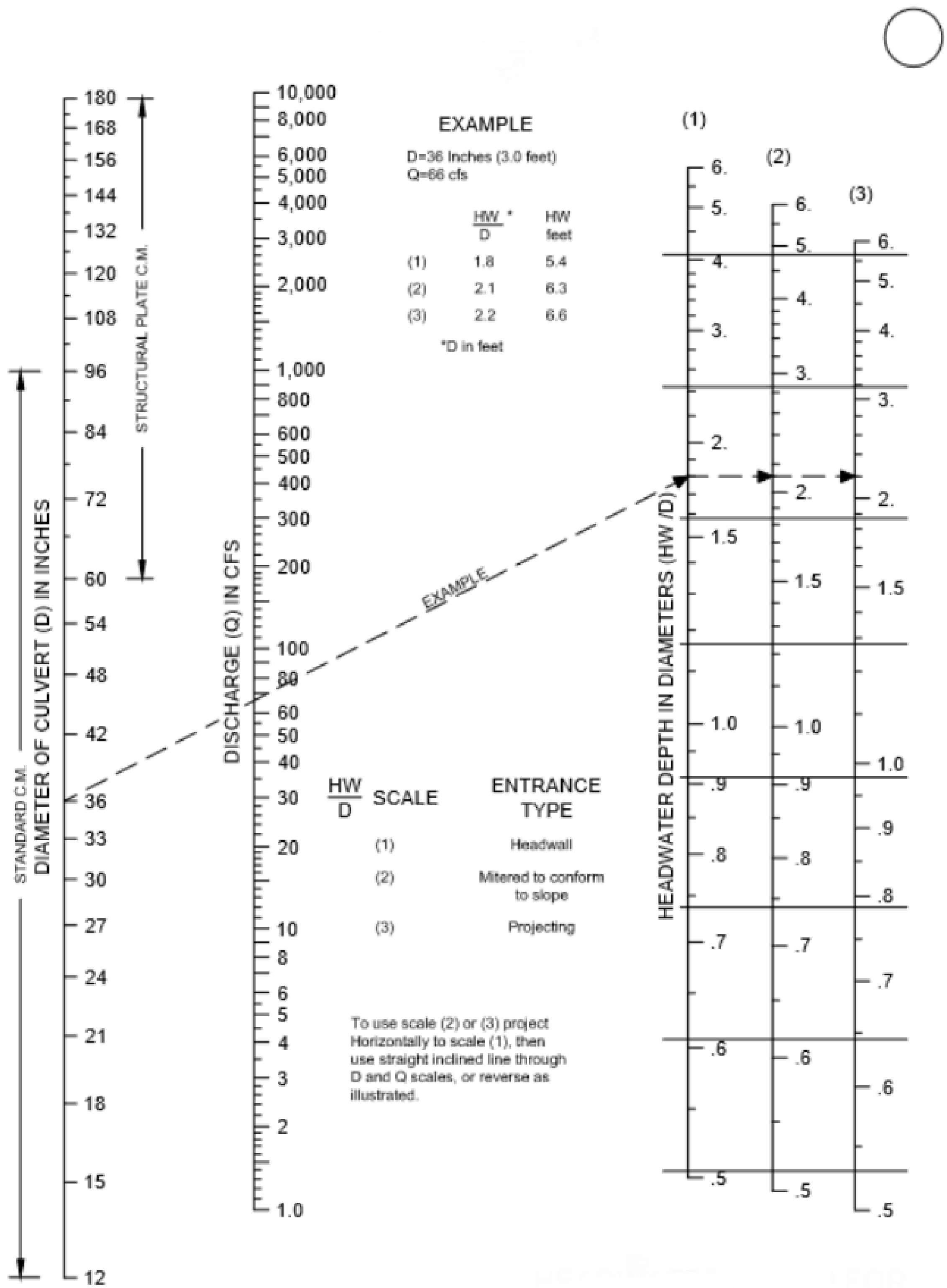
Figure 7.19: Inlet Control Headwater Depth for Concrete Pipe Culverts

(USDOT, FHWA, HDS-5, Chart 1B, 2012)



Note: In example, culvert diameter and discharge are known and solving for headwater depth.

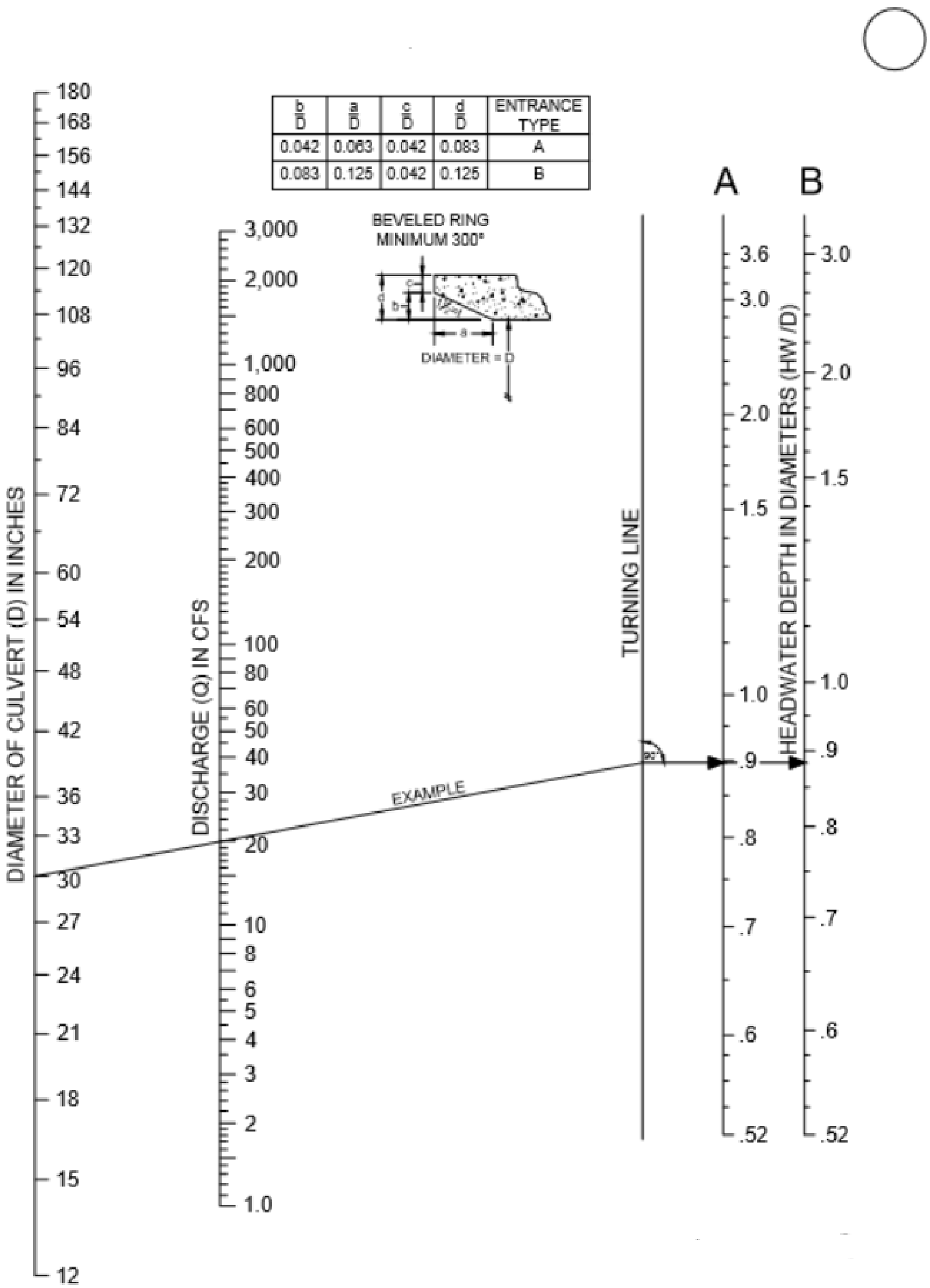
Figure 7.20: Inlet Control Headwater Depth for C.M. Pipe
(USDOT, FHWA, HDS-5, Chart 2B, 2012)



Note: In example, culvert diameter and discharge are known and solving for headwater depth.

Figure 7.21: Inlet Control Headwater Depth for Circular Pipe Culverts with Beveled Ring

(USDOT, FHWA, HDS-5, Chart 3B, 2012)



$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B

Note: In example, culvert diameter and discharge are known and solving for headwater depth.

Figure 7.22: Critical Depth for Circular Pipe

(USDOT, FHWA, HDS-5, Chart 4B, 2012)

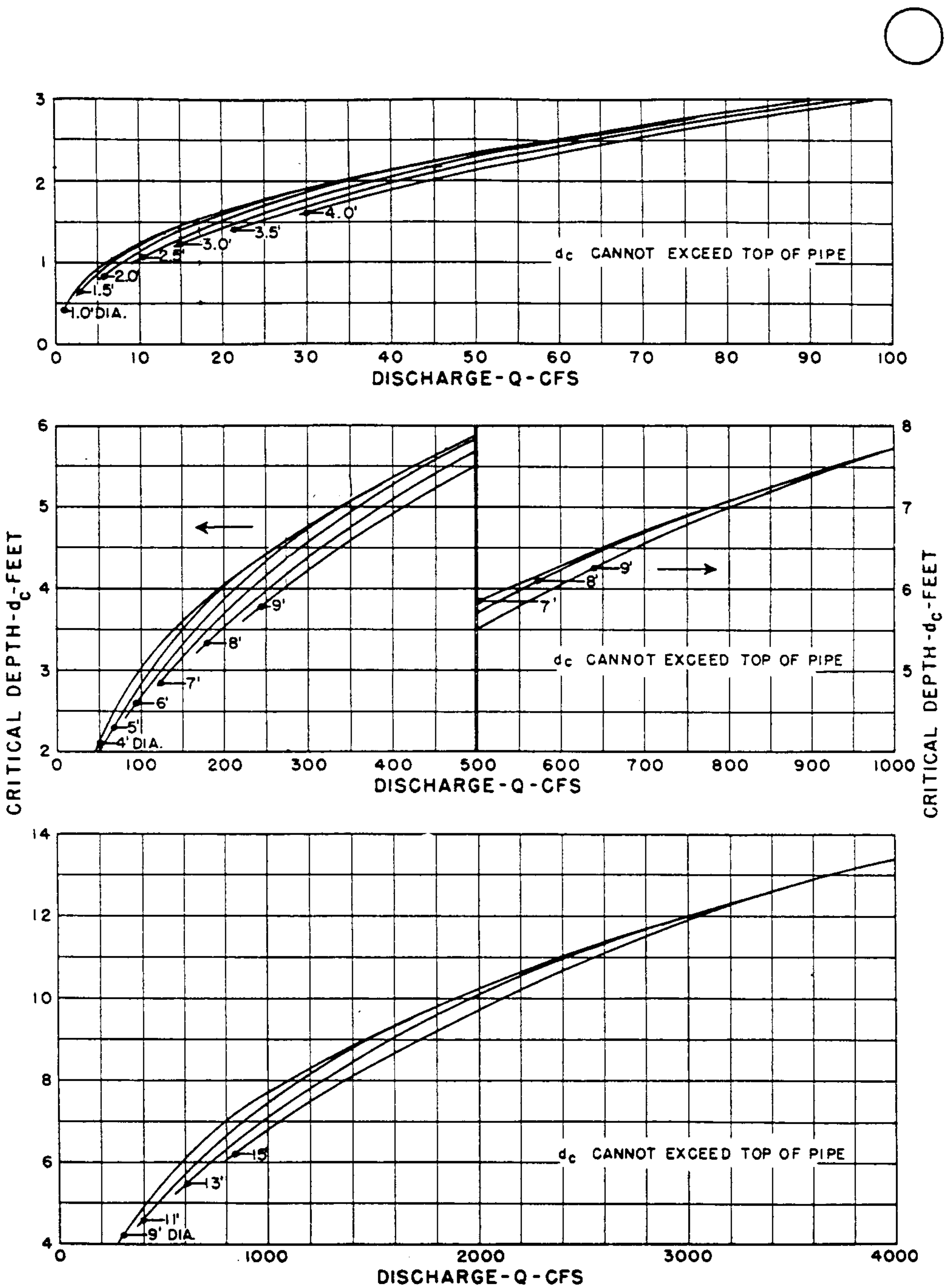
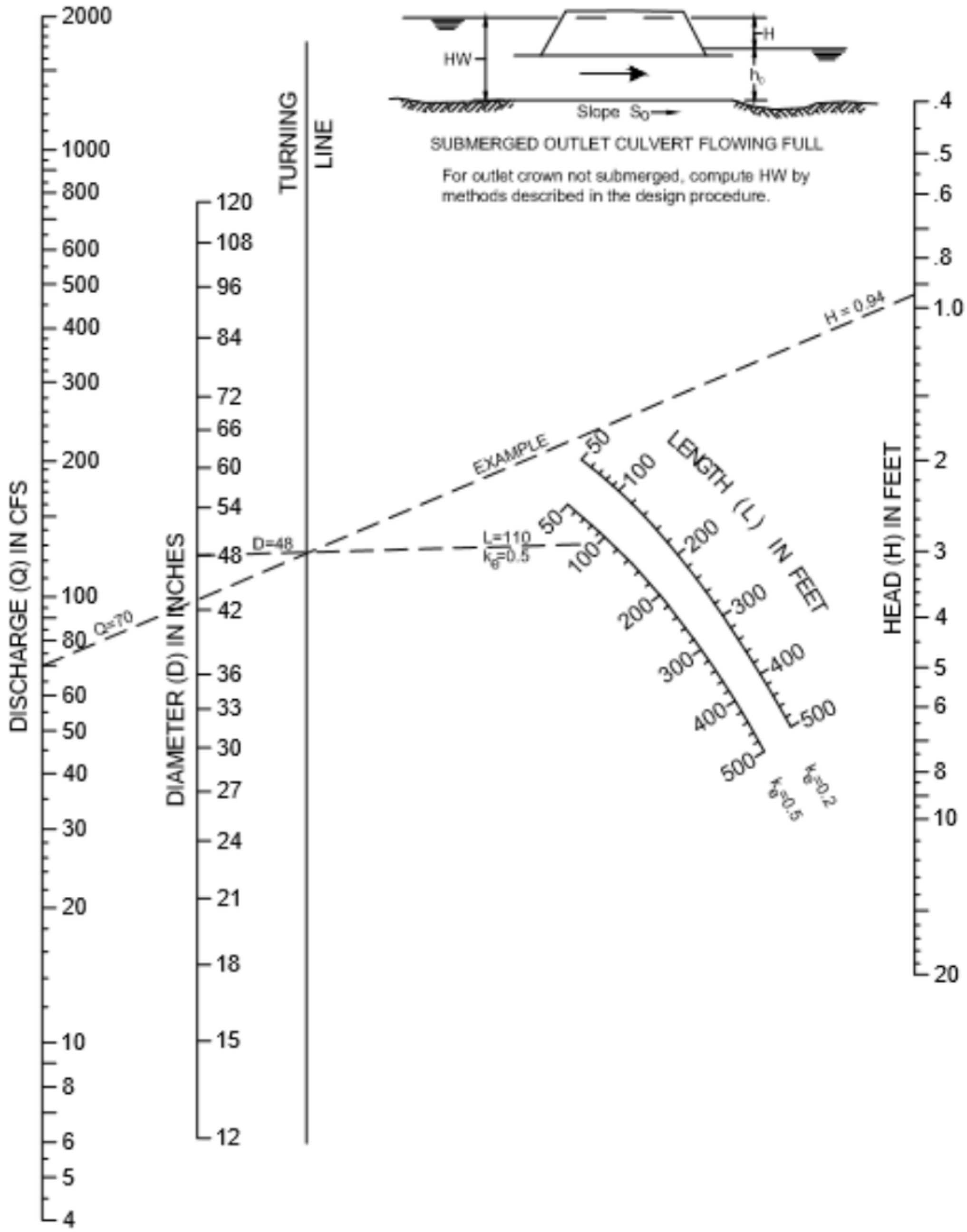


Figure 7.23: Head for Concrete Pipe Culverts Flowing Full

n = 0.012

(USDOT, FHWA, HDS-5, Chart 5B, 2012)

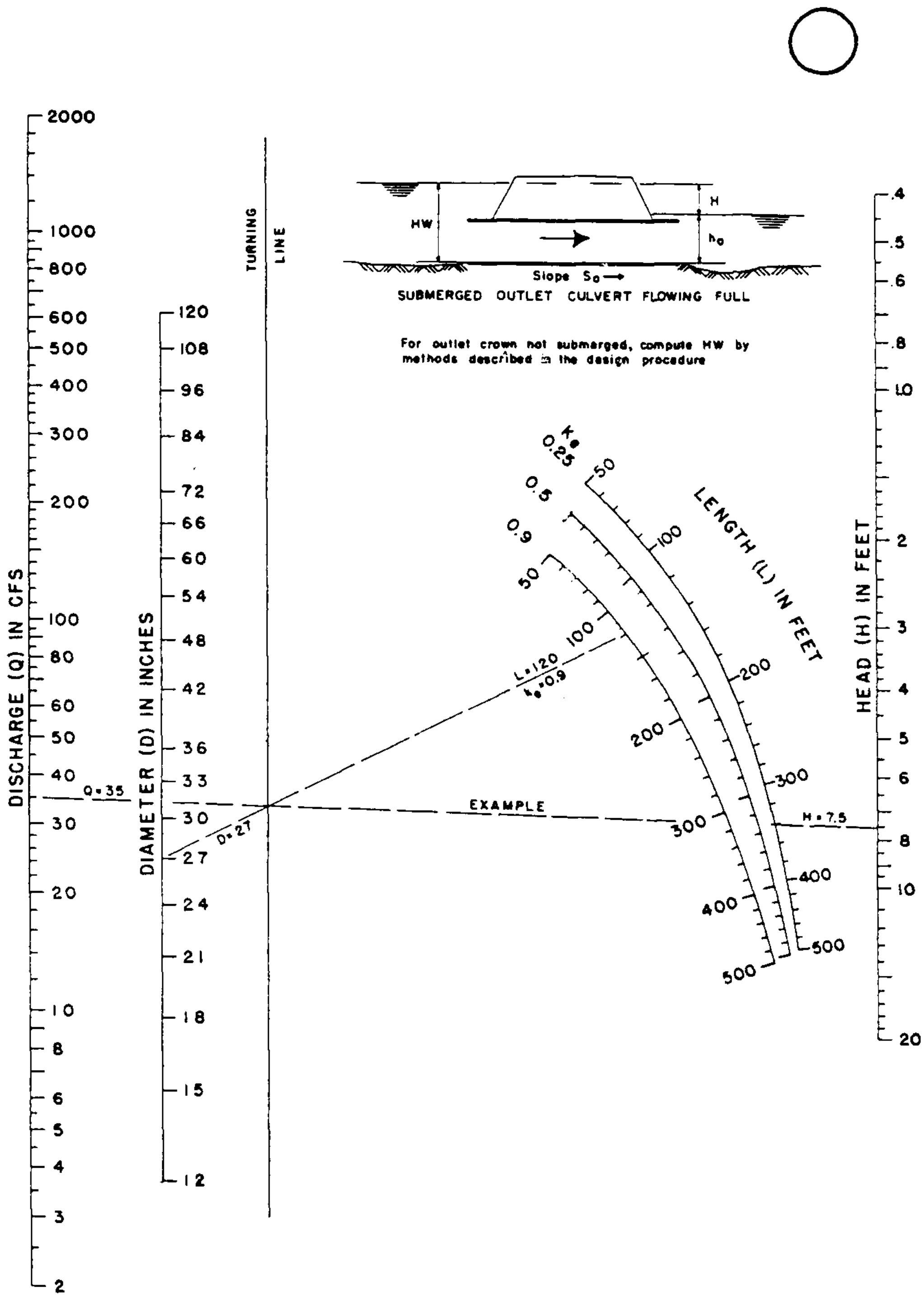


Note: In example, culvert diameter, discharge and culvert length are known. Solving for headwater depth. The District typically uses n=0.013 for RCP, however, this nomograph where n=0.012 is acceptable for use.

Figure 7.24: Head for C.M. Pipe Culverts Flowing Full

n = 0.024

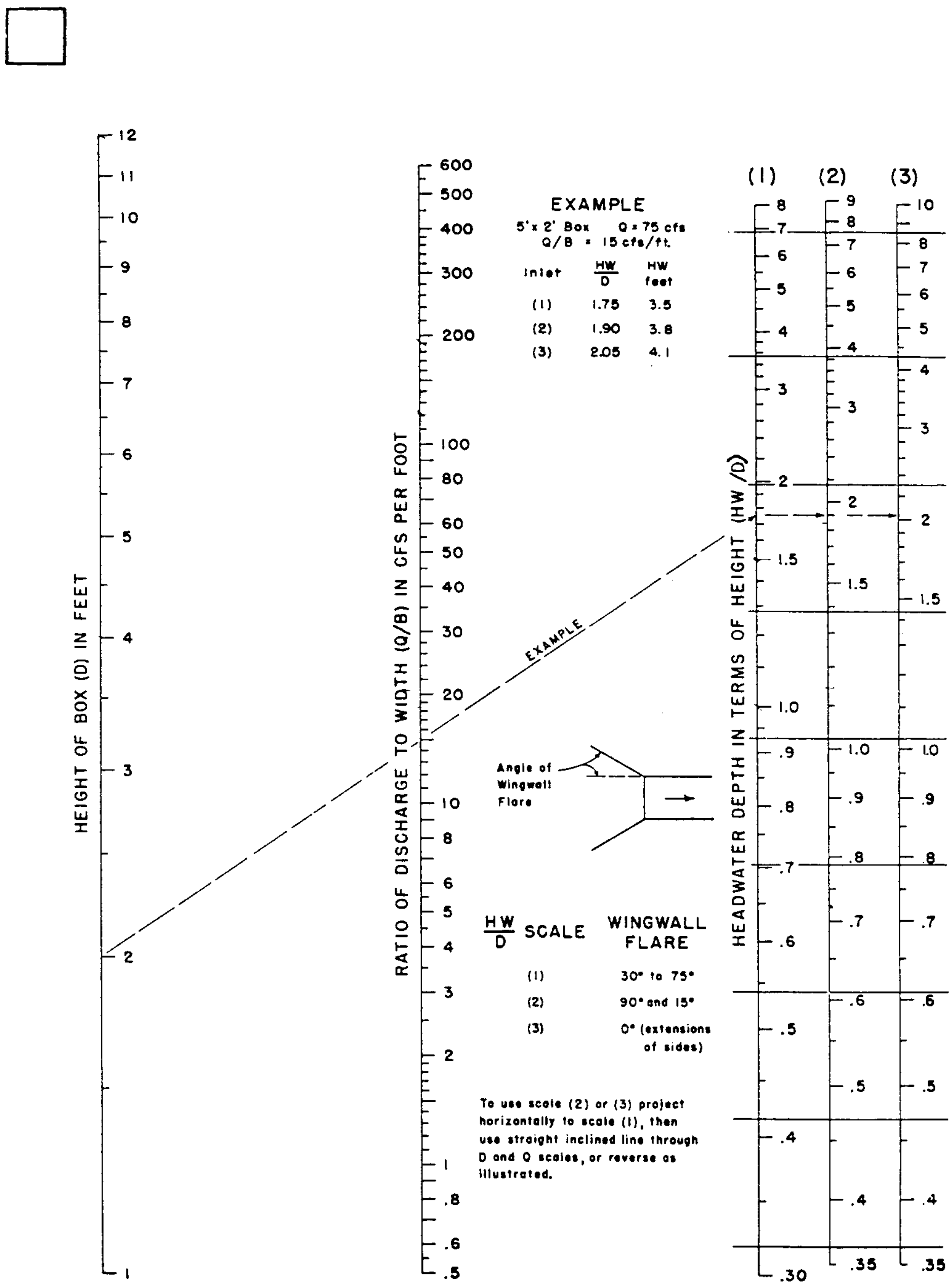
(USDOT, FHWA, HDS-5, Chart 6B, 2012)



Note: In example, culvert diameter, discharge and culvert length are known. Solving for headwater depth.

Figure 7.25: Inlet Control Headwater Depth for Box Culverts

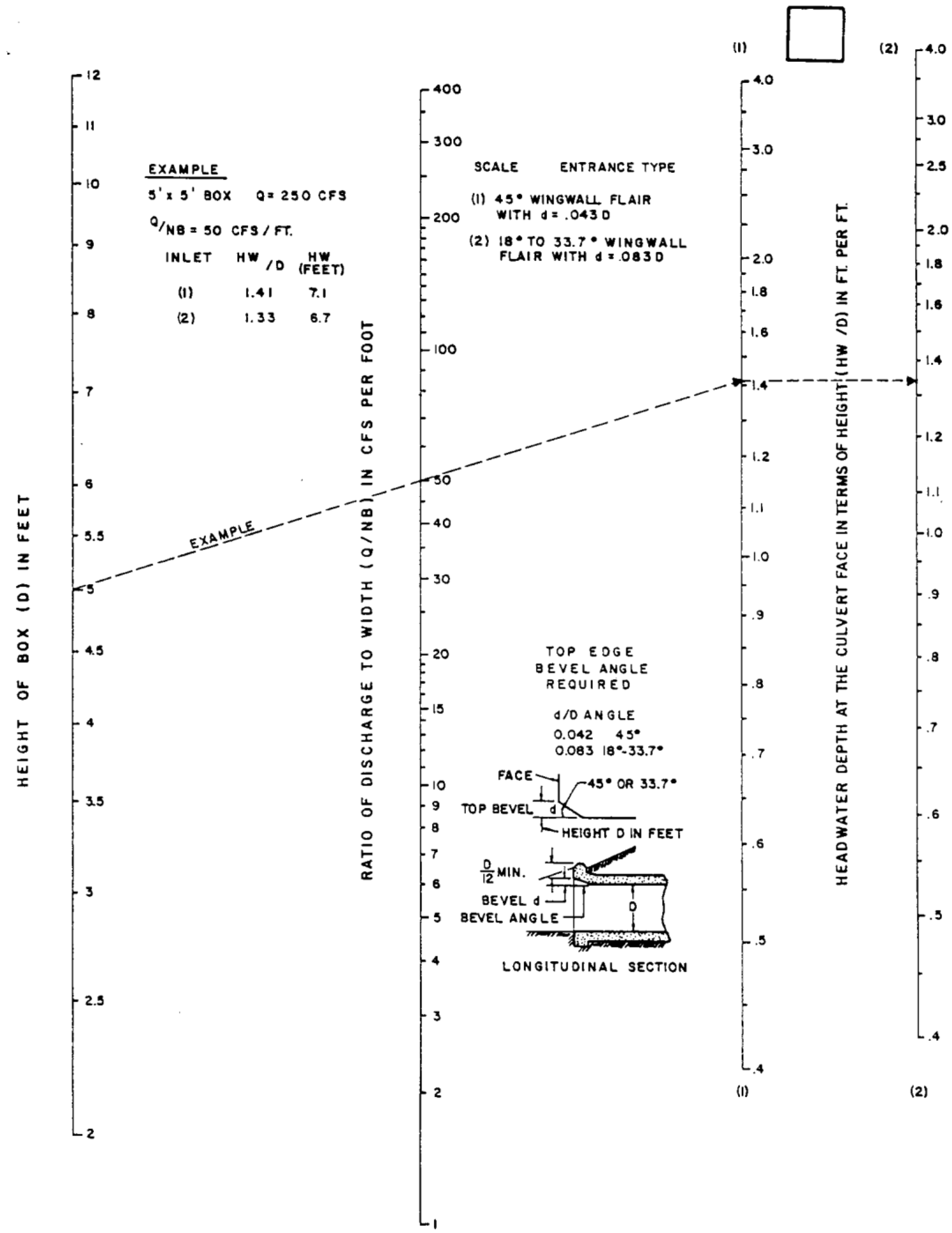
(USDOT, FHWA, HDS-5, Chart 8B, 2012)



Note: In example, culvert height, culvert width, and discharge are known. Solving for headwater depth.

Figure 7.26: Inlet Control Headwater Depth for Rectangular Box Culvert (Flared Wingwalls)

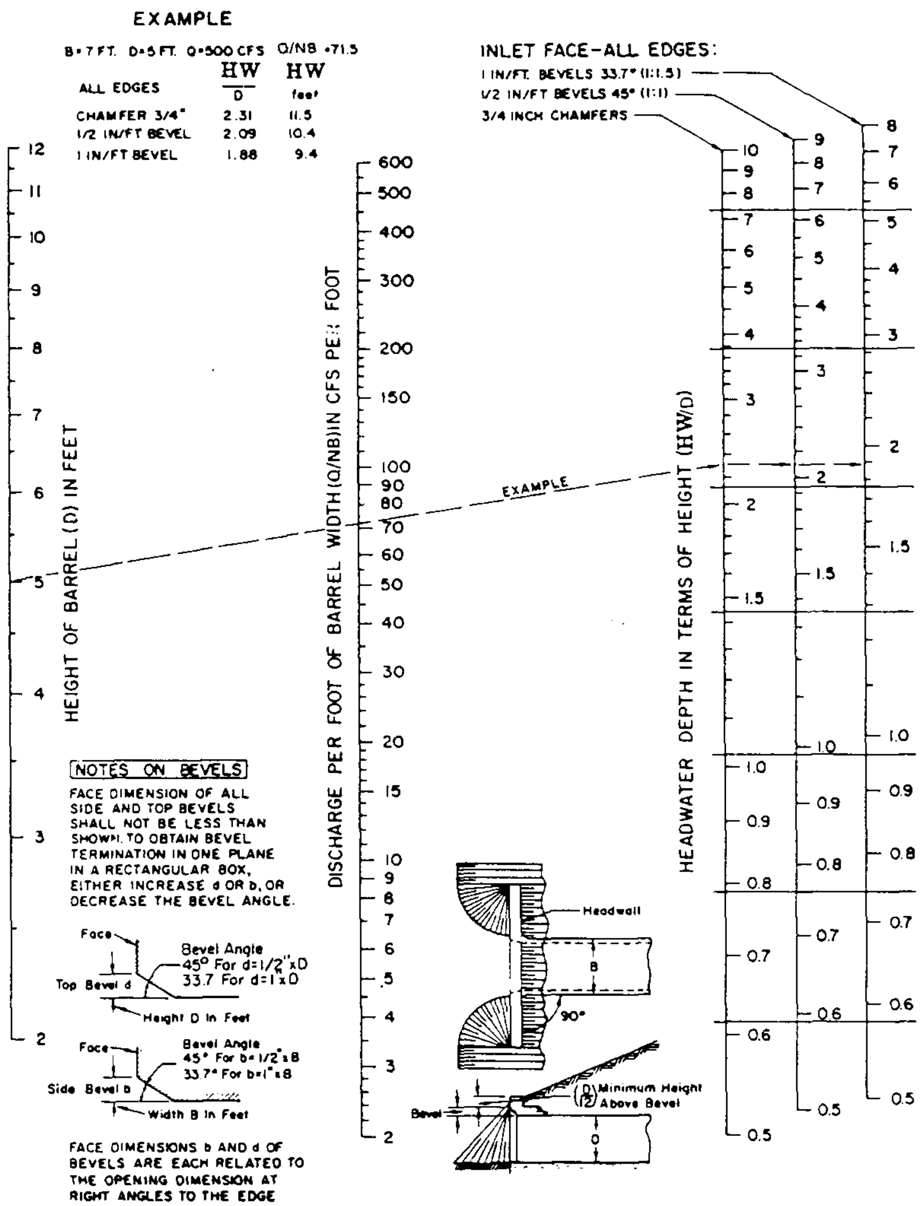
Flare Wingwalls (18^o to 33.7^o, and 45^o) and Beveled Edge at the Top of the Inlet
(USDOT, FHWA, HDS-5, Chart 9B, 2012)



Note: In example, culvert height, culvert width, and discharge are known. Solving for headwater depth.

Figure 7.27: Inlet Control Headwater Depth for Rectangular Box Culvert (90° Headwall)

90^{o} Headwall - Chamfered or Beveled Inlet Edges
(USDOT, FHWA, HDS-5, Chart 10B, 2012)



Note: In example, culvert height, culvert width, and discharge are known. Solving for headwater depth.

Figure 7.28: Critical Depth Rectangular Section

(USDOT, FHWA, HDS-5, Chart 14B, 2012)

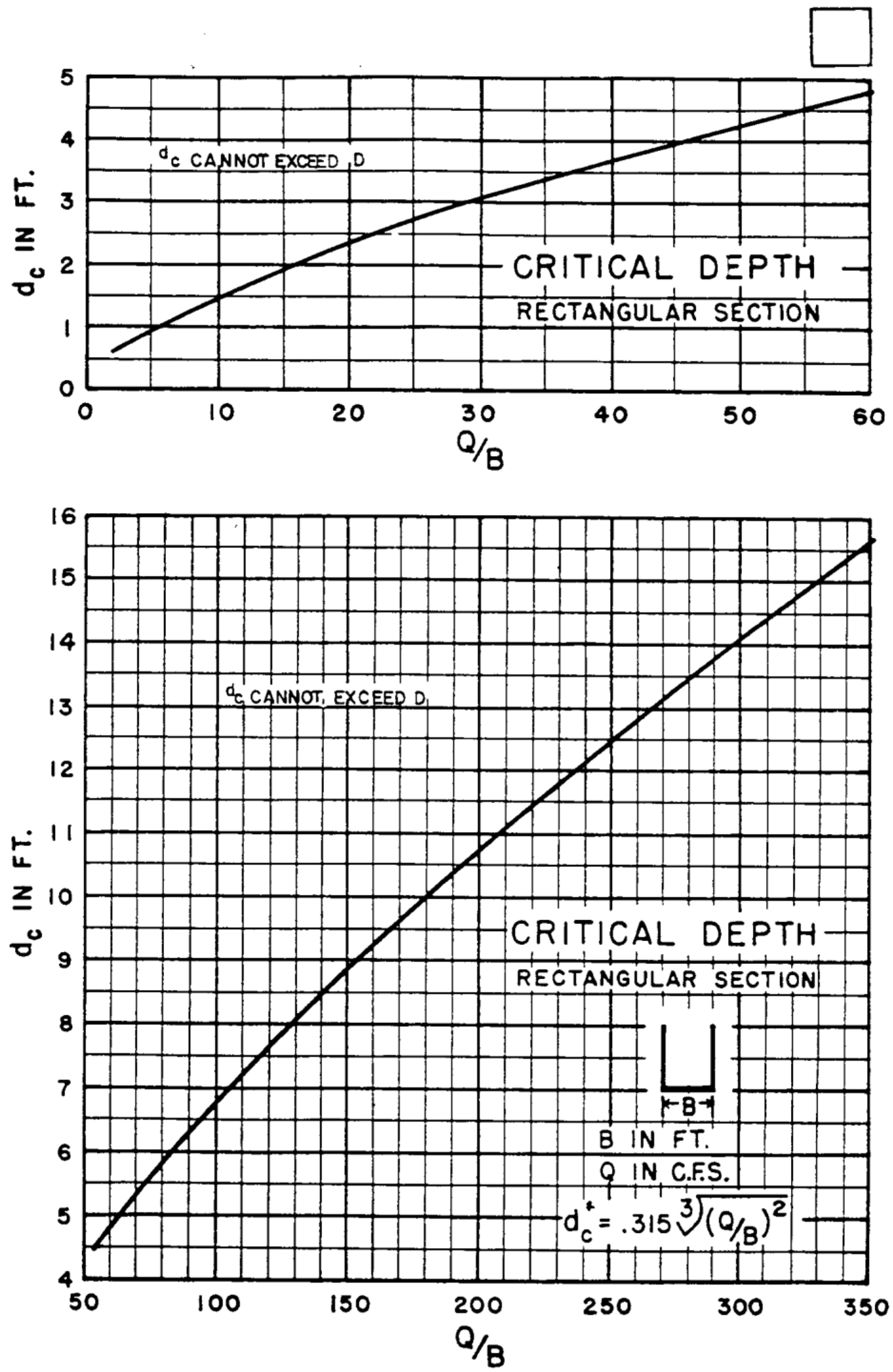
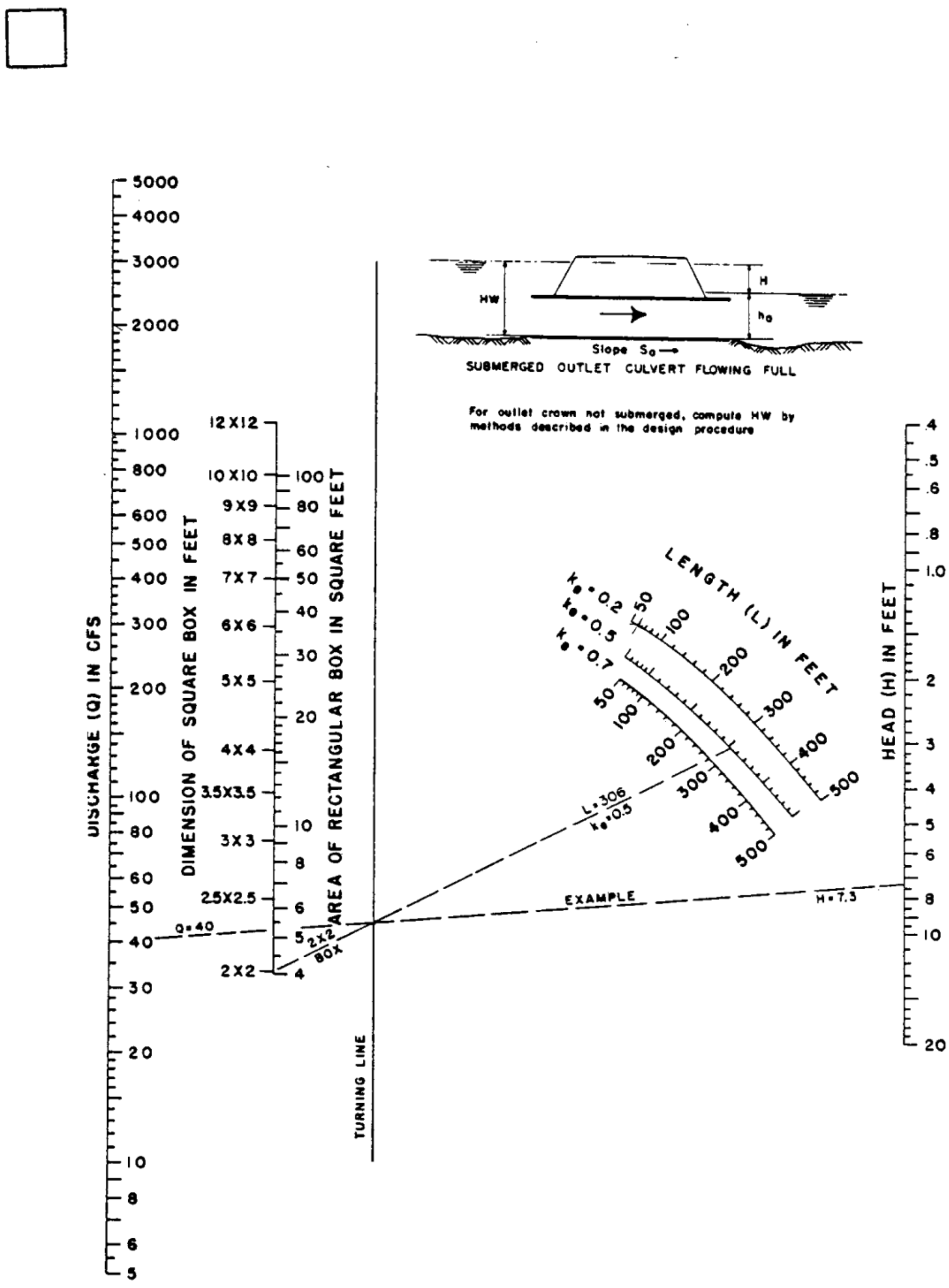


Figure 7.29: Head for Concrete Box Culverts Flowing Full

***n* = 0.012**

(USDOT, FHWA, HDS-5, Chart 15B, 2012)



Note: In example, culvert diameter, discharge and culvert length are known. Solving for headwater depth. The District typically uses n=0.014 for RCB, however, this nomograph where n=0.012 is acceptable for use.

7.4.5 Nomograph Design Examples

The following example problems are from *HDS-5* (USDOT, FHWA, 2012) and illustrate the use of the design methods and charts for selected culvert configurations and hydraulic conditions. The problems cover the following situations:

Example 1: Circular pipe culvert, CMP (standard 2-2/3 by 1/2-inch corrugations) with beveled edge or reinforced concrete pipe with groove end.

Example 2: Reinforced cast-in-place concrete box culvert with square edges and with bevels.

Example 3: Roadway overtopping calculations and performance curve development.

Example 1

A culvert at a new roadway crossing must be designed to pass the 25-year flood. Hydrologic analysis indicates a peak flow rate of 200 cfs. Use the following site information:

- Elevation of stream bed at upstream Culvert Face: 100 ft
- Natural Stream Bed Slope: 1 percent = 0.01 ft/ft
- Tailwater for 25-Year Flood: 3.5 ft
- Approximate Culvert Length: 200 ft
- Shoulder Elevation: 110 ft

Design a circular pipe culvert for this site. Consider the use of a corrugated metal pipe with standard 2-2/3 by 1/2-inch corrugations and a headwall with beveled edges, and concrete pipe with a groove end, projecting. Base the design headwater on the shoulder elevation with a 2-foot freeboard (elevation 108.0 ft). Set the inlet invert at the natural streambed elevation, no drop at the inlet (T=0 in Figure 7.30).

Figure 7.30 represents a completed Culvert Design Form for this example. Notice the headwater depth of 8 feet at the inlet. The designer should verify that backwater from the culvert will not present a hazard to upstream facilities and that flow will not be diverted into another watercourse. An easement may be necessary for ponding on private property. Notice the high estimated outlet velocity of 13.5 fps. The designer should provide outlet erosion control in conformance with Section 7.3.4 or Section 7.3, or investigate other culvert options such as a larger pipe size or multiple smaller pipes. When making this decision, the designer should consider the geometry and allowable velocity of the receiving channel to be sure that the selected pipe or pipes are appropriate given the width and depth of the receiving channel. The design should not result in erosion of the bed, banks, or overbanks of the downstream system.

Note: Figure 7.19, Figure 7.20, Figure 7.22, Figure 7.23, Figure 7.24, and Table 7.1 were used in this example.

Figure 7.30: Example 1 Culvert Design Form
(USDOT, FHWA, HDS-5, 2012)

PROJECT: Example Problem No. 1	STATION: 1 + 00	**CULVERT DESIGN FORM**
	SHEET 1 OF 1	DESIGNER / DATE: WJJ / 7/18
		REVIEWER / DATE: JMN / 7/19

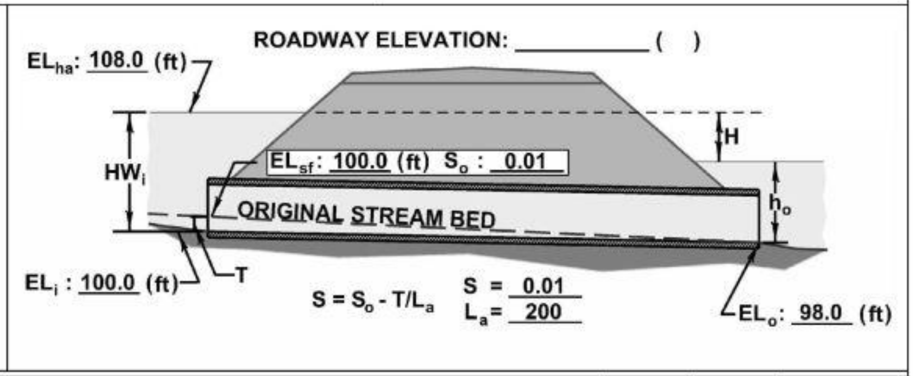
HYDROLOGICAL DATA

See Add'l Shts.

☐ METHOD: Rational
☐ DRAINAGE AREA: 125 Ac. ☐ STREAM SLOPE: 1.0%
☐ CHANNEL SHAPE: Trapezoidal
☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
25	200	3.5



CULVERT DESCRIPTION: / **HEADWATER CALCULATIONS**

MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	INLET CONTROL HW$_i$/D (2)	HW$_i$	T (3)	EL$_{hi}$ (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL$_{ho}$ (8)	Control Headwater Elevation	Outlet Velocity	Comments
C.M.P. - Circ. - 72 in. - bevel 45° in headwall	200	200	0.96	5.76	--	105.8	3.5	3.8	4.9	4.9	0.2	2.6	105.5	105.8	9.0	Try 60" CMP
C.M.P. - Circ. - 60 in. - bevel 45° in headwall	200	200	1.40	7.10	--	107.0	3.5	4.1	4.6	4.6	0.2	6.3	108.9	108.9	11.9	Try 60" Conc.
Conc. Circ. - 60 in. - groove end	200	200	1.36	6.80	--	106.8	3.5	4.1	4.6	4.6	0.2	2.9	105.5	106.8	15.6	Try 54" Conc.
Conc. Circ. - 54 in. - groove end	200	200	1.77	7.97	--	108.0	3.5	4.1	4.3	4.3	0.2	4.7	107.0	108.0	15.3	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

(3) $T = HW_i - (EL_{ha} - EL_{sf})$
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:

High outlet velocity - outlet protection or larger conduit may be necessary

CULVERT BARREL SELECTED:

SIZE: 54 in.
SHAPE: Circular
MATERIAL: Conc. n .012
ENTRANCE: Groove End

Example 2

A new culvert at a roadway crossing is required to pass a 50-year flow rate of 300 cfs. Use the following site conditions:

- EL_{hd} : 110 ft based on adjacent structures
- Shoulder Elevation: 113.5 ft
- Elevation of Streambed at Culvert Face (EL_{sf}): 100 ft
- Natural Stream Slope: 2 percent
- Tailwater Depth: 4.0 ft
- Approximate Culvert Length: 250 ft

Design a reinforced concrete box culvert for this installation. Try both square edges and 45-degree beveled edges in a 90^{o} headwall. Set the inlet invert at the natural streambed elevation, no drop at the inlet (T=0 in Figure 7.30).

Figure 7.31 represents a completed Culvert Design form for Problem No. 2. Notice the headwater depth of 10 feet at the inlet. The designer should verify that backwater from the culvert will not present a hazard to upstream facilities and that flow will not be diverted into another watercourse. An easement may be necessary for ponding on private property. Notice the high estimated outlet velocity of 12.2 fps. The designer should provide outlet erosion control in conformance with Section 7.3.4 or Section 9.3, or investigate other culvert options such as a larger pipe size or multiple smaller pipes. When making this decision, the designer should consider the geometry and allowable velocity of the receiving channel to be sure that the selected pipe or pipes are appropriate given the width and depth of the receiving channel. The design should not result in erosion of the bed, banks, or overbanks of the downstream system.

Note: Figure 7.25, Figure 7.27, Figure 7.28, Figure 7.29, and Table 7.1 are used in this solution.

Figure 7.31: Example 2 Culvert Design Form

(USDOT, FHWA, HDS-5, 1985)

PROJECT: Example Problem No. 2

STATION: 1 + 00

SHEET 1 OF 1

CULVERT DESIGN FORM

DESIGNER / DATE: WJJ / 7/18

REVIEWER / DATE: JMN / 7/19

HYDROLOGICAL DATA

See Add'l Shts.

☐ METHOD: SCS

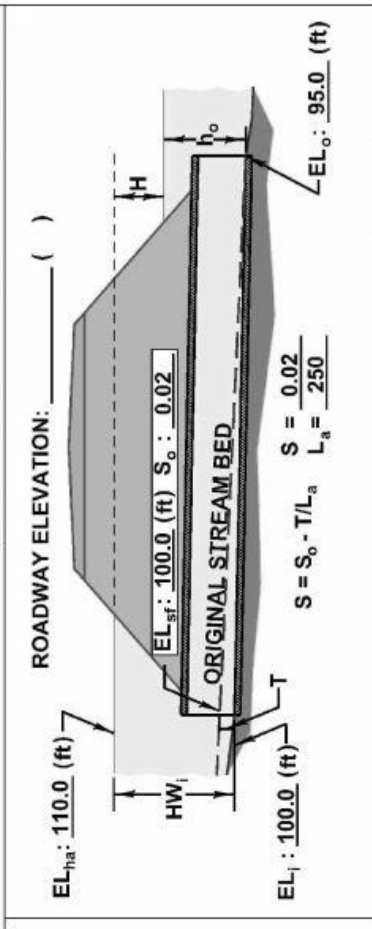
☐ DRAINAGE AREA: 200 Ac. ☐ STREAM SLOPE: 2.0%

☐ CHANNEL SHAPE: Trapezoidal

☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
50	300	4.0



CULVERT DESCRIPTION:

MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	T (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	Control Headwater Elevation	Outlet Velocity	Comments
Concrete - Box - 6' x 5' - Sq. edge	300	50	1.57	7.9	--	107.9	4.0	4.2	4.6	4.6	0.5	3.55	103.2	107.9	21.1	OK try Sm. box
Concrete - Box - 5' x 5' - Sq. edge	300	60	1.91	9.6	--	109.6	4.0	4.8	4.9	4.9	0.5	5.2	105.1	109.6	20.8	Check bevels
Concrete - Box - 5' x 5' - 45° bevel	300	60	1.71	8.55	--	108.6	4.0	4.8	4.9	4.9	0.2	5.0	104.9	108.6	20.8	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

(3) $T = HW_i - (EL_{ha} - EL_{sf})$
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:

5' x 5' box will work with or without bevels. Bevels provide additional flow capacity.

CULVERT BARREL SELECTED:

SIZE: 5 ft x 5 ft

SHAPE: Rectangular

MATERIAL: Conc. n .012

ENTRANCE: Sq. edge - 90° headwall

Example 3

Develop a performance curve for the installation in Figure 7.32 below, including roadway over-topping up to 0.5 feet above the roadway. Use the following dimensions:

Tailwater Channel:	
Flow, cfs	***TW*, ft**
50	101.8
100	102.6
150	103.1
200	103.5
250	103.8
300	104.2
350	104.4

Figure 7.20, Figure 7.22, and Figure 7.24 were used in completion of the Culvert Design Form. Figure 7.33 represents a completed Culvert Design Form for this problem. Figure 7.34 provides the performance curve and roadway overtopping computations.

Figure 7.32: Example 3 Roadway Overtopping and Performance Curve Development

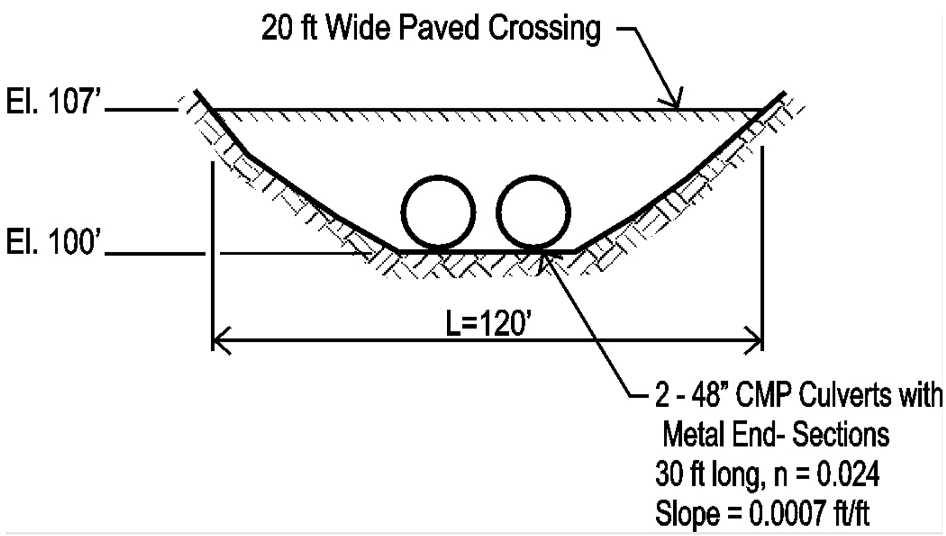


Figure 7.33: Example 3 Culvert Design Form

CULVERT DESIGN FORM

PROJECT: Example
Roadway Overtopping - Performance Curve

STATION: ______ SHEET ______ OF ______

DESIGNER / DATE: ______ / ______
REVIEWER / DATE: ______ / ______

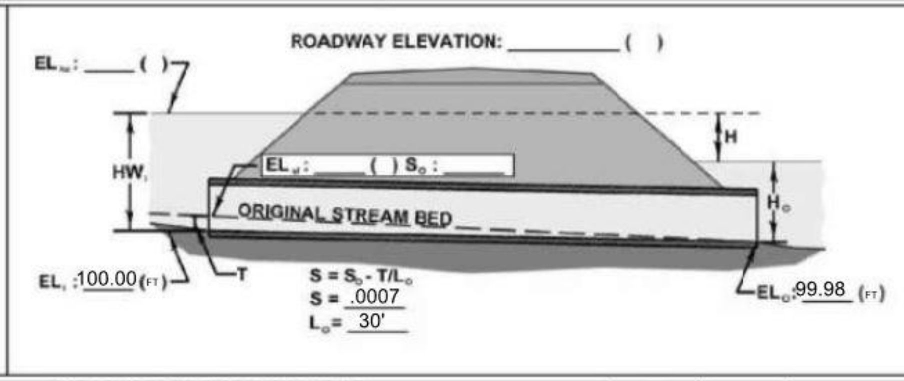
HYDROLOGICAL DATA

See Add'l Shts.

☐ METHOD: ______
☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: 0.0008
☐ CHANNEL SHAPE: Trapezoid
☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

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



CULVERT DESCRIPTION: MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HW i	T (3)	EL hi (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL ho (8)	Control Headwater Elevation	Outlet Velocity	Comments
CMP - Circular 48" - end sect	50	25	0.52	2.08	--	102.08	1.8	1.5	2.75	2.75	0.5	0.22	103.0			
	100	50	0.78	3.12	--	103.12	2.6	2.1	3.05	3.05		0.60	103.6			
	150	75	1.03	4.12	--	104.12	3.1	2.6	3.30	3.30		1.35	104.6			
	200	100	1.30	5.20	--	105.20	3.5	3.0	3.50	3.50		2.40	105.9			
	250	125	1.63	6.52	--	106.52	3.8	3.4	3.70	3.80	▼	3.75	107.5			

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 Ku = 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

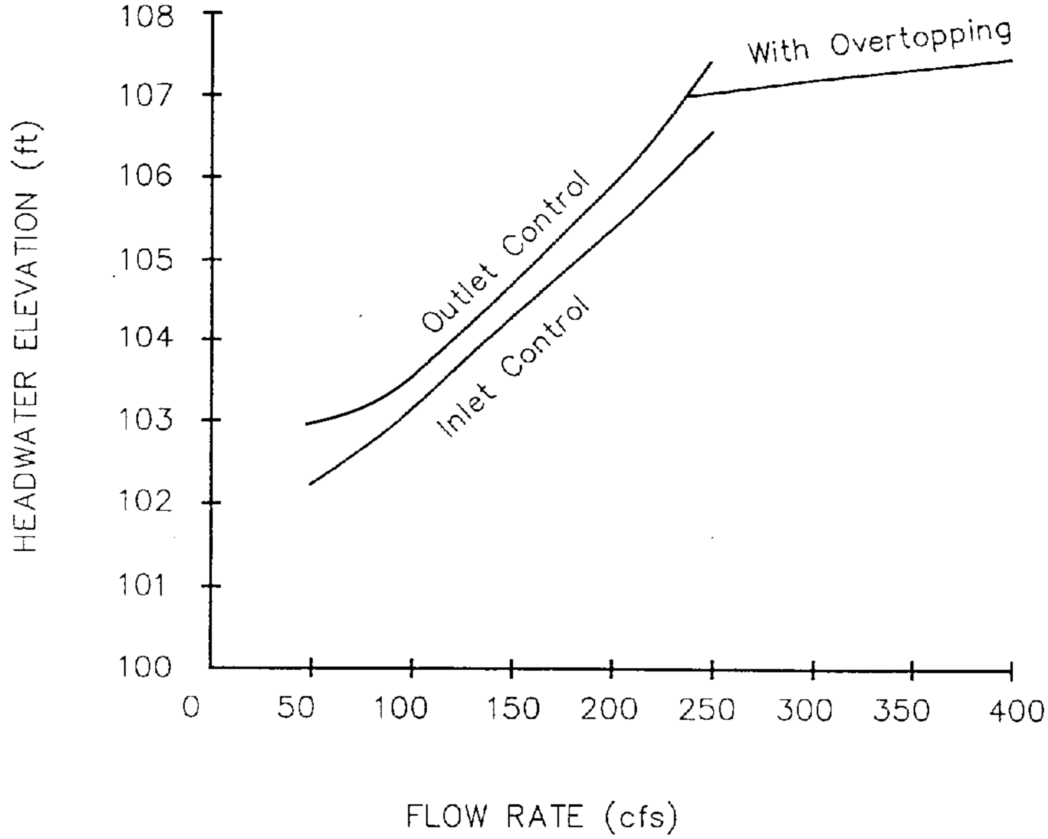
COMMENTS / DISCUSSION:

Used Scale (1) of Figure 6.21 for inlet control comps.
Used Figure 6.25 for outlet control head.

CULVERT BARREL SELECTED:

SIZE: ______
SHAPE: ______
MATERIAL: ______ n ______
ENTRANCE: ______

Figure 7.34: Example 3 Performance Curve and Roadway Overtopping Computations



$$Q_O = K_t C_r L_S (HW_r)^{1.5}$$

HW_r	C_r	K_t	L_S	Q_0	QPIPE		QTOTAL
0.25	2.98	1	120	44.7 cfs	+ 244	=	289
0.50	3.02	1	120	128.1 cfs	+ 250	=	378

7.5 BRIDGES

This Section presents a brief overview of the hydraulic analyses and criteria that must be addressed for bridge crossings over open channels / natural washes. Comprehensive guidelines and criteria for hydraulic analyses of bridge crossings are beyond the scope of this manual. The reader should refer to appropriate texts and technical handbooks for further information on this subject.

Roadways must often cross open channels / washes and sizing the bridge for proper passage of flood waters and protection of the bridge from flood damage is of paramount importance. In general, bridges should be designed to have as little effect as possible upon the flow passing beneath them. Whenever possible, bridges should be designed so that there is no disturbance to the flow whatsoever. Where this is not economically feasible, encroachments into the flow area for construction of bridge abutments and/or piers may be allowable, however, these cause disruption in the flow of water which requires careful analysis for impacts to the floodplain, as well as for protection of the bridge and adjacent structures. Impacts upon channels and floodplains created by bridges usually take the form of increased flow velocities and scour through the bridge opening including a short distance upstream and longer distance downstream of the opening, and upstream ponding due to backwater effects. These impacts can cause flood damage to the channel, adjacent property, and to the bridge structure itself.

A new or replacement bridge should not be permitted to create a rise in the existing water surface elevation, to cause an increase in lateral extent of the floodplain, or to otherwise worsen existing conditions for discharges up to and including the 100-year discharge, unless appropriate measures are taken to mitigate the effects of such increases.

7.5.1 Hydraulic Analysis

The hydraulic analyses of pre- and post-bridge conditions can be performed using a computerized model such as HEC-RAS (for natural or engineered/prismatic channel sections) or WSPGW (for engineered/prismatic channel sections).

Bridge analysis requires meticulous input preparation for proper analysis, and care should be taken to review input data and to examine results thoroughly for reasonableness. Bridge analysis should only be undertaken by an engineer with a solid understanding of hydraulic fundamentals and the software being used.

7.5.2 Hydraulic Design Considerations

When a bridge is proposed across an engineered channel or natural wash that is owned or maintained by the District, the following sections describe certain considerations that must be addressed in the design of the crossing. This list is not exhaustive of all parameters or considerations that the engineer must evaluate, but highlights constraints of particular interest to the District.

7.5.2.1 Freeboard

Freeboard at a bridge is the vertical distance between the design water surface elevation and the low chord of the bridge. The bridge low-chord is the lowest portion of the bridge deck superstructure. The purpose of freeboard is to provide room for the passage of floating debris, to provide extra area for conveyance in the event that debris build-up on the piers reduces hydraulic capacity of the bridge, and to provide a factor of safety against the occurrence of waves or floods larger than the design flood. Freeboard should be provided as required by jurisdictional standards.

The freeboard under the lowest chord of bridge deck (i.e., the soffit elevation) shall be a minimum of 1 foot during the design event. A freeboard of 2 feet is recommended for the 100-year event. Bridges over FEMA accredited levees require additional freeboard per 44 CFR 65.10.b.1.

7.5.2.2 Impact Loads

In certain cases, site conditions or other circumstances may limit the amount of freeboard at a particular bridge crossing. An example would be the replacement of a "perched" bridge across a natural watercourse where major flows overtop the roadway approaches. In general, variances to the minimum freeboard requirement will be evaluated on a case-by-case basis by the jurisdictional agency.

If the required freeboard is unable to be met on a bridge over a District facility, prior approval must be obtained from both the local jurisdiction and the District. In such case, the design of the bridge should consider the possibility of debris and/or flows impacting the bridge. In cases where the bridge has been designed to withstand hydraulic forces of floodwaters and impact from large floating debris, the water surface elevation upstream of the bridge shall maintain a freeboard of at least one foot below the roadway crest and the finished floors of structures within the zone influenced by the bridge headwater. When a bridge crossing increases the existing limits of flooding the project owner shall obtain appropriate documentation from affected property owners as required by governing agency.

7.5.2.3 Piers

Whenever piers are used, they should be oriented parallel to flow. Hydraulic analyses of piers should follow the pressure + momentum method described in Section 4.3.6.

If there is a good possibility of debris collecting on the piers, it may be advisable to use a value greater than the physical pier width to account for debris blockage. Where debris can be expected, bridge piers should be modeled with a debris factor that may vary due to anticipated debris loading, pier geometry, and whether pier nose extensions (debris fins) are utilized. Debris factors typically range from 1-2 feet of blockage modeled on each side of the pier, but larger values may be required where a heavy debris load is expected. Therefore, modeling requirements for debris blockage should be reviewed with the local agency prior to preparing the hydraulic analysis.

7.5.2.4 Supercritical Flow

Supercritical flow is highly sensitive to obstructions or changes in flow area. Such changes can potentially cause large waves, hydraulic jumps, or other conditions that can compromise the structure, adjacent areas, and create other hazards to people and property. For the special condition of supercritical flow within a lined channel, the bridge structure should be designed in a manner to not affect the flow at all. That is, for new / replacement bridges, there should be no projections, piers, abutments, etc. in the channel flow area. The bridge opening should be clear and permit the flow to pass unimpeded and unchanged in cross section.

Supercritical flow profiles through existing bridges with obstructions such as those described above, can be estimated approximately using HEC-RAS or WSPG, however such methods may not fully represent all risks. Advanced knowledge of hydraulics and modeling approaches (beyond the scope of this manual) is required, and in some cases physical modeling studies may be required.

7.5.2.5 Scour

Local pier and abutment scour, contraction scour, and long-term scour must be investigated when designing a bridge over a natural or erodible wash/channel. Refer to Evaluating Scour at Bridges HEC-18 (2012), and LA County Department of Public Works Sedimentation Manual (2006) for guidance and insight into sedimentation and scour.

7.5.3 Safety

Bridges across natural watercourses or channels must be designed with vehicle and pedestrian safety in mind.

7.6 REFERENCES

7.6.1 Cited in Text

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Los Angeles County Department of Public Works, March 2006, Sedimentation Manual

Riverside County Transportation Department (TLMA), 2015, *Improvement Plan Check Policies and Guidelines* https://rctlma.org/trans/Land-Development/Plan-Check/Plan-Check-Guidelines

U.S. Army Corps of Engineers Hydrologic Engineering Center (USACE), 2001a, *HEC-RAS River Analysis System, Hydraulic Reference Manual*.

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——, April 2012, *Hydraulic Design of Highway Culverts-Third Edition, Hydraulic Design Series No. 5*. Publication No. FHWA-HIF-12-026.

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——, 1999, *HY8 Culvert Analysis Version 6.1 Computer Program*.

——, 2006, *Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14*.

7.6.2 References Relevant to Chapter

American Iron and Steel Institute, 1990, *Modern Sewer Design*.

American Society of Civil Engineers and the Water Pollution Control Federation (ASCE), 1969, *Design and Construction of Sanitary and Storm Sewers*.

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——, 1997, *Bridge Scour and Stream Instability Countermeasures, HEC-23*.

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8 OPEN CHANNELS

8.1 INTRODUCTION

8.1.1 Open Channel Defined

"Open channel" systems convey non-pressurized storm water flows that flow by gravity, are exposed to the atmosphere, and may be either natural or engineered. Natural channels typically earthen or naturally vegetated consist of a main flow area and adjacent overbank areas. Engineered channels may have a variety of cross-sectional geometries as they are used for many different applications varying in scale from a modest roadside ditch to a large flood control facility measuring hundreds of feet wide.

8.1.2 Scope of Chapter

This chapter presents technical standards and design criteria for the evaluation and design of natural and engineered open channels and discusses design standards for various channel types that might be encountered in Riverside County. This manual provides only the minimum hydraulic standards for channel evaluation and design. More detailed explanations and further information are available from the technical resources listed at the end of this chapter. Readers are strongly encouraged to review the reference list and consider adding some of those publications to their design library.

The Open Channel chapter contains these general sections:

- Section 8.3 - General considerations for open channel drainage planning, such as route layout, freeboard, toedown, and hydraulic analysis considerations and limitations
- Section 8.5 - 8.9 - Design guidelines for various types of open channels

8.1.3 Channel Design Limitations

Rigid Boundaries - This manual assumes that all channel boundaries are rigid, i.e., the channel cross section and gradient is unaffected by erosion/scour for all flows. In this respect, this chapter is limited to channels where erosion, transportation, and deposition of sediment are not critical design considerations, or where the channel system has been engineered in such a manner to mitigate such concerns. Channels requiring consideration of non-rigid boundaries, such as where they are subject to long term degradation or aggradation and/or lateral meandering are beyond the scope of this manual. References in Section 8.13 provide some additional resources with respect to these topics.

Froude Number - Recommendations in this chapter address only channels designed to sustain subcritical or mildly supercritical flow regimes. Highly supercritical flows, such as flows through steep chutes downstream of spillways, introduce design factors and considerations that are outside of the scope of this manual. Section 4.1.2 contains discussion of the calculation of the Froude number and the determination of flow regime. For the purposes of this manual, Froude

Numbers between 1.0 and 2.0 are generally considered mildly supercritical. Froude numbers above this range may require special methods and considerations.

Fluid Flow - The methods and parameters in this chapter are specific to the flow of stormwater or clear water runoff by gravity flow. These methods do not apply to other fluids. It is understood the stormwater flows will have some minerals or other materials entrained within the fluid that can affect parameters such as density, however this manual assumes that stormwater will behave similar to clear water. Conditions where there is significant entrainment of sediment of 'mud flows' are not covered in this manual.

Designers are strongly encouraged to design projects such that channels stay within the guidelines presented in this Section, unless alternative analytic procedures, guidelines, etc. can be substantiated in a manner acceptable to the governing jurisdiction.

8.2 TYPES OF OPEN CHANNELS

Open channels can be categorized as either natural or engineered. Natural channels include all watercourses that are carved and shaped naturally. The focus of this manual is on engineered channels constructed by human efforts, and that are deigned to convey stormwater flows. Such channels may be lined or unlined depending on whether a lining is required for the channel to be stable for the design flow conditions. Options for engineered channel linings vary with the shape of the channel, the velocity of the water, soil conditions, material availability, aesthetics, economic, environmental, sociological, and regulatory factors, maintenance requirements, and compatibility with existing improvements. Typical channel linings include concrete, soil cement, rock (riprap or cobble). Engineered channels may also be unlined, in which case the channel is earthen. Some unlined channels are vegetated with plants, such as grass, trees, or shrubs, to provide some level of erosion protection for the channel banks and bottom. Channel linings can be used alone over the entire channel section or in combination with other linings or vegetation.

8.2.1 Natural Watercourses

A natural channel is a watercourse formed originally by natural processes. It is not engineered, but has evolved or geologic timescales in response to the full spectrum of runoff events that the watershed has experienced. In some cases, a natural channel may continually change its position and shape as a result of hydraulic forces acting on its bed and banks, changes to sediment load, geologic activity, or other natural phenomena. Changes in runoff patterns due to development (hydromodification) can impact natural channels. Section 10.5 of this manual describes methods to ensure that hydromodification is mitigated for the protection of downstream natural watercourses. The District typically does not provide maintenance of natural watercourses.

Wherever feasible, natural channels shall be kept undisturbed, and improvements set sufficiently away from any areas subject to flooding or erosion hazards. Pad elevations within new developments shall be placed at least one foot above 100-year water surface elevations for adjacent natural channels.

Modifying (or 'channelizing') a natural watercourse can create impacts on the hydrologic

cycle and potentially on natural habitats and species both upstream and downstream of the project site. Wherever possible natural channels should be preserved in their existing condition and projects designed to position improvements away from any flooding impacts.

If a natural channel cannot be integrated into a project undisturbed, the order of preference for engineered open channels in subcritical flow conditions is:

1. Earthen channels with vegetated bottoms and side slopes (preferably native vegetation), adding periodic hardened (concrete or riprap) grade-control structures if needed to control velocity;
2. Compound channels consisting of earthen vegetated bottoms with riprap, soil cement, or concrete lined side slopes;
3. Fully riprap lined channels; and
4. Fully concrete-lined channels. Where supercritical flow conditions occur, in engineered channels, only acceptable structurally sound channel linings such as concrete or grouted riprap are recommended.

See Section 8.4 for design considerations when developing in or near a natural watercourse.

Figure 8.1: Natural Channel

8.2.2 Earthen and Vegetated Channels

This category includes engineered earthen channels, including both naturally vegetated (hydroseeded with native plant mix) and landscaped (either with grass or shrubs and trees) and sometimes irrigated. In either case, establishment of vegetation on the bed and banks either by hydroseeding or planting can be critical to the ability of an unlined channel to resist erosive forces.

Earthen unlined channels must be designed for subcritical flow regimes and slow non-erosive velocities. Normally, these channels are relatively small and do not require low flow channels. However, if used for larger channels, defined herein as having a bottom width over 50 feet, a low flow channel is required to facilitate ease of maintenance. See Section 8.2.8.

In some cases, the low flow channel can be used as an area where vegetation is left to grow. Often in urbanized areas, continual flows from sources such as irrigation runoff, will cause dense growth within an earthen channel low flow. In such a case, the n-value for the channel, particularly the low-flow area must be updated accordingly. In other cases, a concreted low flow channel may be used to control meandering, limit growth due to anthropogenic flows, or to provide a location where sediment deposition can be focused and subsequently cleaned out while the channel and surrounding watershed stabilizes after development. The low flow design should be checked for the effect that less frequent storms may have on sediment or scour, in terms of maintenance and aesthetic implications. Earthen and Vegetated engineered channels will require maintenance and repair over the lifetime of the facility both to restore lines and grades (remove sediment deposition, repair erosion, etc.), remove trash and debris, and to maintain the intended hydraulic capacity. Accordingly, such a channel must be designed with this maintenance in mind. The project must obtain not only any regulatory permits that allow for initial construction of the channel, but also permits that allow for this ongoing maintenance.

District Standard Drawing CH324 depicts an example of an unlined earthen channel.

Figure 8.2: Earthen Channel

Grass lined channels are engineered channels with grass or other short-stemmed vegetation lining the bottom and sides. In a semi-arid environment such as Riverside County, there is not enough natural rainfall to maintain a grass lined channel without irrigation. While the District typically does not maintain channels that include irrigation systems, we have worked in tandem with other agencies in situations where the District maintains the channel lines and grades, while the partner public agency maintains the landscaping and irrigation.

Refer to Sections 8.3 and 8.5 for design considerations of earthen and vegetated channels.

Figure 8.3: Grass Lined Channel

8.2.3 Concrete Lined Channels

Concrete lined channels are rectangular or trapezoidal channels in which reinforced concrete is used for lining of the channel banks and/or bottom. They are used primarily where right of way is limited, or velocities exceed the allowable conditions for other channel linings. Concrete lined channels may be designed for either subcritical or supercritical flow.

The anticipated structural loads, clearance requirements of the reinforcing steel, and, in some cases, anticipated bed load delivered from the upstream watercourse will dictate the thickness of the concrete lining. Where large bedload material or small material with high velocities are expected the concrete aggregate size may also be adjusted to help mitigate degradation of the concrete. Weep holes and subdrains are required to prevent uplift pressures from hydrostatic forces in the event saturated soil conditions occur. Where channel sides are sloped (not vertical), reinforced cut-off walls are required at the top of the lining. Some of these concepts are illustrated in District Standard Drawings CH326 to CH328. Designers are cautioned against copying these details directly without first evaluating the design conditions for their specific project. Refer to Sections 8.3 and 8.6 for design considerations of concrete lined channels.

Figure 8.4: Concrete Lined Channel

8.2.4 Rock Riprap Lined Channels

Rock riprap lined channels are channels in which riprap (typically pieces of angular granitic rock) is used for lining the channel banks and bottom. Riprap may be utilized in a loose or grouted fashion, with the effect of grouting being that smaller pieces of rock when grouted together with concrete act as larger pieces of rock and thus have greater resistance to movement. Riprap is a popular choice for channel lining because the material availability and initial installation costs in Riverside County are often less than alternative methods for preventing channel scour. However, the design engineer must be mindful that there may be other costs associated with using riprap that may increase typical installation costs, for example, transport costs where the source for the proper size of rock is far from the project, or placement methods that require the installer to place one rock at a time instead of dumping. Additionally, riprap installations require periodic inspection and maintenance due to the nature of riprap to shift, or in some case migrate downstream, over time.

Riprap lined channels may be permitted in areas where right of way limitations, high velocities, or changes in channel geometry, such as bends, drops, or transitions, preclude the use of vegetated earthen channels. Riprap lining is typically also used immediately downstream, and in some cases upstream, of hydraulic structures. Section 8.7 provides specific design criteria for riprap lined channels.

Where shearing forces are greater than those that can be resisted by loose riprap, grouted riprap may be used. Grouted riprap channel sections often used as transitions from reinforced concrete structures to loose riprap lined channels because the increased roughness of the rock helps dissipate energy before transitioning to the loose riprap lining.

Figure 8.5: Rock Lined Channel

8.2.5 Soil Cement

Soil cement is used to stabilize banks from erosion, in otherwise natural or soft bottom channels. It is most commonly used where the banks must be steeper than would be allowable with natural soil or rock, such as where right of way is limited or near bridges as channel bank slopes transition to meet steep or near vertical bridge abutments. Soil cement can be constructed easily on slopes of 0.5H:1V. Depending on the availability of suitable soil, and space for onsite batching, this can be more environmentally friendly and economical than constructing reinforced concrete slopes.

Soil cement linings are composed of a thick layer (typically 8 feet measured perpendicular to flow but may be greater, see Figure 8.14) of unreinforced granular soil mixed with Portland cement. Soil cement is more subject to weathering and abrasion than riprap or concrete and, therefore, in certain environments, may have more long-term maintenance requirements than these other hardened linings. Soil cement can withstand relatively high velocities for short periods of time. Refer to Sections 8.3 and 8.8 for design considerations of soil cement channels.

Figure 8.6: Soil Cement Side Slopes

8.2.6 Soft Bottom, Bank Lined Channels

Bank lined (revetment) channels are a type of channel stabilization where the banks are lined (e.g., concrete, rock, or soil cement) but the channel bottom remains in a natural state with minimal grading. This type of channel is typically utilized where channel velocities are such that scour or lateral meandering is likely and needs to be limited, but environmental considerations preclude the use of a lined channel bottom. Bank lined channels are designed so that the lined bank extends below the channel bottom to at least the anticipated scour depth. In cases where the lining cannot be adequately extended to the scour depth, the use of a launchable riprap toe may be allowed based on Corps of Engineers EM 1110-2-1601 or FHWA HEC-11. The soft bottom of the channel must be designed with similar n-value, low flow, maintenance, and permitting considerations as described for Earthen and Vegetated channels. District Standard Drawings CH323 and CH325 depict examples of soft-bottom, bank lined channels. Refer to Sections 8.3 and 8.9 for design considerations of soft bottom, bank lined channels.

Figure 8.7: Soft Bottom, Bank Lined Channel

8.2.7 Other Types of Channel Lining

The District generally prefers the aforementioned channel types for facilities that are to be maintained by the District. There are however many alternative engineered channel lining products available to the design engineer from various manufacturers. These include, for example, gabion boxes and mattresses, cable-stayed articulated concrete block mattresses (ACB), interlocking concrete blocks, turf reinforcing mats, concrete revetment mats, and various types of synthetic fiber linings. These alternative channel linings may be allowed on a case-by-case basis with prior consultation with the governing agency. The governing agency may reject the proposed lining system in the interests of operations and maintenance, prior difficulties with a given product, or if they believe the product being proposed will not adequately protect the public safety.

For those linings not discussed in this manual, supporting technical documentation will be required to support use of the desired lining. Some of the items that should be addressed include:

- Structural integrity of the proposed lining
- Interfacing/transition between different linings
- The maximum velocity and/or shear stress under which the lining will remain stable
- Potential erosion and scour problems, and mitigation measurers
- Anticipated operations and maintenance requirements
- Long term durability of the product
- Repair methods in the event of damage to the lining
- Past case history (if available) of the lining system in similar applications
- Potential groundwater mitigation issues (i.e., weepholes, underdrains, etc.)

8.2.8 Low Flow Channels

Per District Standard Drawings CH323, CH324, and CH325, a low flow channel is used when the main channel base width exceeds 50 feet. The overall purpose of a low flow channel is to concentrate low flows and/or small storm runoff into a specific portion of the invert.

There are two types of low flow channels: 1) earthen, and 2) concrete. When concrete low flow channels are used, riprap may be installed along the outer edges of the concrete low flow channels to prevent erosion adjacent to the concrete. This riprap is especially needed at bends.

Often in urbanized areas, continual flows from sources such as irrigation runoff, will cause dense growth within an earthen low flow. In such a case, the n-value for the low flow area must be updated accordingly. A concreted low flow channel may be used to control meandering, limit growth due to anthropogenic flows, or to provide a location where sediment deposition can be focused and subsequently cleaned out while the channel and surrounding watershed stabilizes after development. The low flow design should be checked for the effect that less frequent storms may have on sediment or scour, in terms of maintenance and aesthetic implications.

Within a detention basin, low flow channels also permit the incorporation of recreational amenities into a wide channel bottom by confining low discharge flows to a specific part of the overall flood control channel. Many large drainage basins have small base flows resulting from irrigation returns, treatment plant effluent, or urban cooling water. In addition, the most frequent storms produce runoff considerably smaller in magnitude than the storm for which the channel was designed. In the long term, these high frequency, low magnitude flows may deposit considerable amounts of sediment in a channel. Sediment deposition can cause redirection of flow toward the channel banks resulting in bank erosion. Additionally, a meandering low flow channel in channels where the bottom has other uses can be problematic from an O&M standpoint. It is noted that a meandering low-flow channel may be seen as beneficial in restoration projects, or where environmental project goals require that a channel, once constructed, be left alone. In these cases, omission of a low-flow channel must be discussed and agreed upon in the design phase with the governing agency.

8.3 GENERAL DESIGN CRITERIA FOR OPEN CHANNELS

8.3.1 <u>General</u>

This Section presents general hydraulic design standards that are applicable to all engineered channels: Section 8.4 through Section 8.9 discusses additional design standards for the specific types of engineered channels. The specific requirements for a particular type of channel may be stricter than the general design criteria outlined in this Section.

Good design practice requires that several issues be addressed in the analysis and design of an open channel project. Water surface profiles must be computed for all channels during final design and clearly shown on the final drawings. Computation of the water surface profile should use methods described in Section 4.4. These computations must account for losses due to changes in velocity, drop structures, bridges, and culverts, etc.

8.3.2 <u>Horizontal Alignment</u>

The horizontal alignment of a safe and economical drainage system should be one of the first steps in the land development process. An open channel that is well planned and designed should incorporate several features as follows.

8.3.2.1 Follow Natural/Existing Drainage Patterns

Generally, drainage systems should follow the existing and natural drainage patterns. For new development projects, drainage system requirements may determine the character of the development, and often dictate the layout of streets and lots. Attention to drainage requirements during the first phases of planning will result in better land use decisions and lower maintenance costs.

Collection/Discharge Points: Unless special exception is made by the governing agency, all engineered channels must begin and end where, historically, runoff has flowed. Unless there are overriding reasons to the contrary and permission is obtained from impacted property owners, runoff conveyed by the channel must be collected and discharged at the same point and in the same manner as that prior to construction. At both ends of a new channel, geometry must tie to existing geometry, and the flow velocity and water surface elevation must not be greater than before the channel was constructed, without permission of all impacted property owners. This also means that transitions meet the natural channel must be constructed within the project site unless permission is granted otherwise.

Alignment with Structures: The channel should be aligned with any structures, such as bridges and culverts, such that flow approaches the structure in-line with the path water will take through the structure. Sharp angles should be avoided. See also Section 7.2.2.11.

8.3.2.2 Angle Points

Horizontal alignment changes of two degrees or less may be accomplished without the use of a circular curve for subcritical flow designs. Curves must be used for supercritical flow designs, no matter the degree of change in horizontal alignment.

Uncontrolled local runoff should not be allowed to enter the channel; rather, it should be collected and discharged into the channel through a structure specifically designed for that purpose. In all cases, the issue of wet and dry weather safety should be a paramount consideration in route determinations.

8.3.2.3 Curves

Sharp and/or closely spaced curves should be avoided.

Under subcritical flow conditions, the minimum radius of a curved channel, measured to the channel centerline, is recommended to be greater than or equal to three times the width of the water surface assuming the channel is conveying the design storm flow rate. That is:

$$r_c \geq 3T \tag{8.1}$$

Under supercritical flow conditions, the recommended minimum radius is:

$$r_c = \frac{4V^2T}{gy} \tag{8.2}$$

where:

r_c = recommended minimum radius of curvature at channel centerline (ft)
V = mean channel velocity (ft/s)
T_W = water surface top width for the design water depth at channel centerline (ft)
g = gravitational acceleration (ft/s^2)
y = depth of flow (ft)

Curved channel alignments shall have sufficient freeboard above water super-elevation calculated in accordance with Section 8.3.9.

8.3.3 Vertical Alignment

8.3.3.1 Longitudinal Channel Slope

Maximum and minimum channel slopes are dictated by permissible velocity and Froude number limits, as presented in Sections 8.3.4.4 and 4.1.2.2. **Under any case, the minimum channel slope for District channels is 0.1 percent.** Where the natural topography is too steep to achieve the maximum permissible velocity for a given lining, drop structures may be utilized to flatten the channel slope and slow velocities.

8.3.3.2 Grade Control

Whenever designing an earthen, vegetated, or soft bottom engineered channel, a key design element, including during conceptual layout, is establishing whether or not grade control is required to limit or prevent long term degradation of the channel.

By definition, grade control is any natural or man-made structure within a channel that limits or prevents downward vertical erosion of the channel bed. Examples can include natural or constructed rock out-croppings, drop structures, and transverse cutoff walls.

Where a natural or soft-bottom channel will be used, it should be assessed for the risk of long-term scour degradation. Where degradation may occur, grade control can be constructed to either prevent such degradation, or ensure that such scour does not compromise adjacent, or upstream improvements. Constructed grade controls are either 1) embedded below the channel invert to limit future anticipated scour from migrating upstream or laterally risking the channel or other infrastructure, or 2) they are constructed as at-grade drop structures that are designed to force a channel profile that is stable, preventing such long-term scour.

Grade control and channel slope are interrelated. In the design of grade control structures, the stability of the study reach must be assessed in context of the equilibrium of the entire system. When designing artificial channels, the designer needs to assess the stability of the reach immediately downstream of the segment under design. If there is evidence of ongoing degradation in the downstream system, grade control structures may be required to protect the proposed channel.

Grade control structures must be designed so that the downstream side of the structure extends below the channel invert to the total scour depth, which includes local scour due to the grade control structure itself, long-term scour, general scour, and perhaps other scour components. Local scour is the scour occurring over a limited area due to local flow conditions, typically from the grade control structure causing acceleration of or turbulence within the flow, which create vortexes that remove the surrounding sediments. Long-term scour results from stream bed profile changes that occur from aggradation and/or degradation. General scour is lowering of streambed elevation (also called bed degradation) over long reaches due to head cuts and changes in hydrology controls (such as dams, sediment discharge, or river geomorphology).

Scour analysis is beyond the scope of this manual, refer to L.A. County Department of Public Works Sediment Manual for guidance and insight into sedimentation and scour. When properly designed, the use of grade control structures can result in a stable channel that also enhances public safety by mitigating high velocities.

8.3.3.3 Profile of Channel Wall

The vertical alignment of the channel shall be set with consideration of the required depth of the channel (considering hydraulics, superelevation, freeboard, etc.). If the vertical alignment is set too shallow relative to the adjacent topography, the top of the channel may be higher than the adjacent natural topography.

This creates two potential problems that either must be managed, or avoided by lowering the channel profile:

- Side Drainage: If the channel wall is higher than the adjacent topography it will block natural surface drainage from entering the channel from adjacent land, potentially causing flooding outside the channel or on adjacent property. Lowering the channel profile can resolve this issue. Where lowering the profile is not possible, special provisions for accepting this surface runoff must be provided in the design. An example would be providing a swale parallel to the channel to collect these flows and direct them to a drop inlet.
- Levee Condition: If the actual water surface within the channel is higher than adjacent ground, a levee condition can be created. Levees have significant additional requirements that are beyond the scope of this manual, including special engineering requirements for the embankment, inspections, and certifications to avoid the higher water surface from being mapped as a remnant floodplain outside the channel.

8.3.4 Hydraulic Criteria

8.3.4.1 Hydraulic Calculation Method

Open Channels shall be designed in accordance with the Water Surface Profile methods described in Section 4.4 of this manual.

8.3.4.2 N-Value Selection Criteria

Typical values of roughness coefficients for channels of a uniform and consistent material type across the cross section (such as engineered lined or earthen channels) are given in this Section. For naturally vegetated or composite systems such as natural or urban floodplains, see Section 4.2.6.6. See Table 8.1 for each material and/or construction method listed for the range of n values to be used.

Table 8.1: n-value Section for Various Design Criteria

Facility Type	Design Criteria	n-value Selection
Earthen or Vegetated Channel	Peak Velocity	Minimum applicable value from Table 4.2 to represent prismatic "clean" condition before any erosion or vegetative growth
	Depth / Capacity / Freeboard / Shear Stress	Maximum applicable value from Table 4.2 representing the maintained condition of the channel that is permitted by regulatory agencies[2]
Concrete Lined	Peak Velocity	Minimum applicable value from Table 4.3
	Depth / Capacity / Freeboard / Shear Stress	Maximum applicable value from Table 4.3
Rock Lined (loose)	All	Calculated pursuant to Equation (4.31) and Table 4.4
Rock Lined (grouted)	Peak Velocity	Calculated from HEC-15 with a 20% reduction in n-value
	Depth / Capacity / Freeboard / Shear Stress	Calculated pursuant to Equation (4.31) and Table 4.4

[2] By default, the n-value used for the design of earthen channels shall assume vegetative growth will populate within the channel. If the design n-value will assume a condition where such future vegetation is mowed or removed, the project proponent MUST obtain regulatory maintenance permits, acceptable to the maintenance entity, that allow such post-construction maintenance to occur. In the absence of such permits, the channels must be designed assuming the vegetative growth is left in place.

Facility Type	Design Criteria	n-value Selection
Natural Channels / Floodplains	Peak Velocity	Value calculated pursuant to Section 4.2.6.6 using aggressive (lower) assumptions appropriate to the site
	Depth / Capacity / Freeboard / Shear Stress	Value calculated pursuant to Section 4.2.6.6 using conservative (higher) assumptions appropriate to the site

$S_{f\text{-}avg}$ should be determined using Manning's equation as described in Section 4.2.6.

The hydraulic design of a channel should be based upon the maximum n-value anticipated during the life of the structure when looking at channel capacity, since a higher n-value results in a conservative estimate of flow depth. The maximum n-value for a particular channel material as listed in Table 4.2 is representative of the end of design-life condition and should be used to determine water surface elevation. Conversely, when considering the effects of peak velocity, supercritical flow, hydraulic jumps, and energy dissipators, hydraulic design should be based on the minimum n-value and, therefore, the maximum velocity of flow in the channel. The minimum n-values as listed in Table 4.2 represent newly constructed conditions. Both newly constructed (minimum n-value) and end of design life (maximum n-value) conditions should be analyzed when designing an open channel.

8.3.4.3 Minimum Velocity

Open channels must be designed to balance a number of factors including maintaining a sufficient minimum velocity to minimize sediment deposition and nuisance ponding, while not exceeding the maximum velocity that the chosen channel linings can sustain.

While designing channels to have tranquil, subcritical flow is generally desirable, very low velocities encourage sedimentation and plant growth, which decreases channel carrying capacity and can promote nuisance ponding and vector/mosquito issues. To minimize such issues, a minimum permissible velocity of 2-3 fps should be maintained.

The minimum permissible velocity must be compared to the minimum flow velocity that may occur in the channel reach being designed. The designer must perform two checks:

- Perform the Water Surface Profile calculations (Section 4.4) using the applicable Manning's n-value for 'capacity' as described in Table 4.2.

- Evaluate flows of smaller, more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions of the design flow rate (e.g., one-half, one-quarter, and further if necessary).

Another design philosophy is to design a steeper channel such that the sediment is moved through the channel. Because of higher velocities with this design approach, engineered linings may be required for the channel banks, bottom, and other structures.

8.3.4.4 Maximum Velocity

The design of open channels shall be governed by maximum permissible velocity of the channel lining. This design method assumes that a given channel lining will remain stable up to a given velocity, provided that the channel is designed in accordance with the standards presented in this manual. Maximum permissible velocity is influenced by characteristics such as soil cohesive properties, material grain/rock size, presence of vegetation, etc. Table 8.2 presents the maximum permissible velocities for various natural and engineered linings. Where channel velocities are in excess of the permissible velocity, the design must be modified to either reduce the velocity, or an alternative lining must be selected, concrete or riprap drop structures or other suitable velocity control design features can be utilized.

The maximum permissible velocity must be compared to the maximum flow velocity that may occur in the reach being designed. The designer must perform two checks:

- Perform the Water Surface Profile calculations (Section 4.4) using the applicable Manning's n-value for 'peak velocity' as described in Table 4.2.
- Evaluate flows of smaller, more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions of the design flow rate (e.g., one-half, one-quarter, and further if necessary). Additional geotechnical and geomorphological investigation and analyses may be required for earthen, vegetated, or soft-bottom channels to verify that the channel will remain stable based on the maximum design velocities.

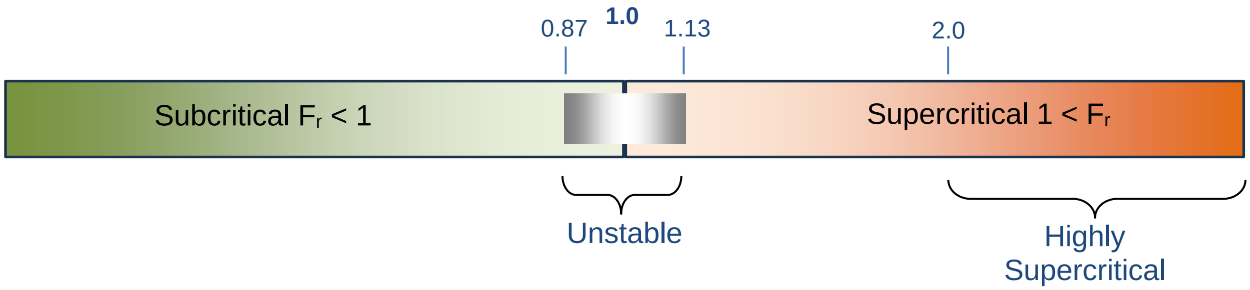
Table 8.2: Maximum Permissible Velocities for Lined and Unlined Channels

Material/Lining Type	Permissible Velocity (1) (ft/sec)
Natural and Engineered Unlined Channels	
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	2.0
Ordinary Firm Loam	2.5
Fine Gravel	2.5
Stiff Clay (very colloidal)	3.75
Graded, Loam to Cobbles (noncolloidal)	3.75
Graded, Silt to Cobbles (noncolloidal)	4.0
Alluvial Silts (noncolloidal)	2.0
Alluvial Silts (colloidal)	3.75
Coarse Gravel (noncolloidal)	4.0
Cobbles and Shingles	5.0
Shales and Hard Pans	6.0
Silty Clay	2.5
Sandy Silt	2.0
Clay	6.0
Fully Vegetated Channels	
Unreinforced Vegetation/Grass Lined(2)	6.0
Engineered Lined Channels(3)	
Loose Riprap(4)	8.0 - >20.0
Grouted Riprap	25.0
Soil Cement	15.0
Concrete(5)	40.0

(1) Maximum permissible velocities listed here are basic guidelines; higher design velocities may be used if appropriate technical documentation is provided. This may include shear stress calculations, or in the case of engineered proprietary linings, manufacturer information.
(2) Fully vegetated with dense perennial vegetation such as grasses covering all soil, with irrigation as necessary to ensure that the vegetation will always exist.
(3) Engineered linings shown are generally acceptable for channels intended for District maintenance.
(4) Applicable velocity will vary by size of loose riprap. Size loose riprap per Section 8.7.
(5) See Section 6.2.8 for additional cover requirements.

8.3.4.5 Froude Number Limit

Drainage systems should be designed such that the Froude Number does not fall between 0.87 and 1.13 for any significant length of the channel in order to maintain stable flow conditions. For a given flow rate, adjustments in the Froude Number can be achieved primarily by adjusting the channel slope, or by changing the cross-sectional size of the system, which will affect the velocity and hydraulic depth of flow in that drainage system.



Supercritical channels typically have high velocities that may require extra considerations such as engineered channel linings. When the Froude Number is greater than 2.0, the system is considered highly supercritical. Due to safety concerns resulting from excessively high velocities intractable hydraulic forces, air entrainment, and the creation of roll and/or standing waves, it is recommended to avoid Froude Numbers greater than 2.0 except for carefully designed concreted-lined channels.

8.3.4.6 Channel Lining Requirements

Engineered channel linings are required whenever the flow is supercritical and when velocity exceeds the permissible velocity of the native soil. Where multiple lining types may be suitable, selection of a channel lining will need to balance other important criteria such as cost, material availability, maintenance, environmental impact, and acceptability to the governing and maintaining entity. All vegetated and engineered channel linings must be constructed and installed in accordance with applicable agency standard drawings, industry practice, as well as the requirements in this manual.

8.3.4.7 Hydraulic Capacity

All open channels shall be designed to safely contain and convey the runoff from a design flow rate as required by the Governing Agency. All new open channels that will be maintained by the District, shall be designed for the 100-year storm event calculated by the methodology prescribed by the current version of the *District's Hydrology Manual*.

Water surface profiles must be computed for all channels during final design and clearly shown on the final drawings. Computation of the water surface profile should use methods described in Section 4.4 and the n-value criteria in Section 4.2.6 for determining capacity of the system. These computations must account for losses due to changes in velocity, drop structures, bridges, and culverts, etc.

8.3.5 Side Slopes

Channel side slopes shall be limited as follows when designed in accordance with this manual:

Table 8.3: Channel Maximum Side Slope

Channel Type	Maximum Side Slope*
Earthen/Vegetated	4H:1V
Riprap	2H:1V
Concrete Trapezoidal Channel	1.5H:1V
Soil Cement	0.5H:1V

*Steeper slopes than shown above may be approved on a case-by-case basis with engineering calculations demonstrating the slope/structure will be stable for all applicable loading and geotechnical considerations. See the applicable Design Guideline Section. Approval of steeper slopes will also consider factors beyond structural, such as maintenance and safety implications.

8.3.6 Bottom Width

The selection of the channel bottom width is primary dictated by channel capacity but shall also consider other factors such as: possible wetland mitigation requirements, constructability, maintenance and access, multi-use purposes, and the width of the low flow channel (if necessary). The recommended minimum bottom width is 8 feet in order to be drivable.

To allow for future maintenance and replacement of channel bank protection for soft bottom, bank lined channels, a routine maintenance zone is established in the channel invert near the banks as shown on District Standard Drawing CH323 and CH325. Potential wetland mitigation areas of multi-use purposes with maintenance restrictions shall be kept outside the routine maintenance zone.

Table 8.4: Channel Minimum Bottom Width

Channel Type	Minimum Bottom Width (ft)
Concrete Sides and bottom	8
Riprap Sides and bottom	8
Concrete or riprap sides, soft bottom	8
Soil Cement	8
Earthen	8

8.3.7 Freeboard

Freeboard is the additional height of a flood control facility (e.g., channel, levee, or embankment) measured above the design water surface that is provided as a factor of safety. Freeboard is required to ensure that the desired degree of protection shall not be reduced by factors unaccounted for, such as surface waves from wind, debris, etc., that may affect the facility's ability to contain the design storm, but that are not required to be specifically analyzed in design.

Freeboard shall be added to the water surface profile calculated pursuant to Section 4.4 including:

- Use of the maximum Manning's roughness coefficient expected during the lifetime of the channel, as described in Table 8.1 for the 'Capacity' design criteria.
- Full consideration of gradually varied flow profiles and rapidly varied flow impacts (jumps, piers, junctions, etc.)

Table 8.5: Freeboard Requirements

Category	Condition	Freeboard Requirement
Subcritical Channels	Straight	1.0 ft
	Curved	1.0 ft + Superelevation (Section 8.3.9)
Supercritical Channels (including reaches with hydraulic jumps)	Straight	2.0 ft + Roll/Standing Waves (Sections 8.3.8 and 8.3.10)
	Curved	2.0 ft + Superelevation (Section 8.3.9) + Roll/Standing Waves (Sections 8.3.8 and 8.3.10)
	$1.0 < Fr < 1.13$	Maximum of: • Straight or Curved freeboard as applicable The sequent depth for the flow (Section 4.2.5)
Culverts	N/A	See Section 7.2.2.7
Bridges	N/A	See Section 7.5.2.1
Reinforced Concrete Box Storm Drains	Flowing Open (not pressurized)	1.0 ft from top of box. If freeboard is not met, force calculations to assume sealed/pressurized.

This manual only describes Riverside County's minimum freeboard requirements for open channel design. Major drainageways involving road crossings or other types of crossings, streams that the Federal Emergency Management Agency (FEMA) has mapped as Special Flood Hazard Areas, or facilities that interface with Caltrans facilities might have significantly different freeboard requirements. For instance, freeboard for levees must meet FEMA freeboard requirements (3, 3.5, or 4 feet minimum depending on location relative to end of levee and to other structures). Refer to 44 CFR Section 65.10: Mapping of Areas Protected by Levee Systems (USGPO, 2000).

8.3.8 Slug Flow and Roll Waves

Roll waves also known as slug flow are intermittent surges on steep slopes that will occur when the Froude Number is greater than 2.0 and the channel invert slope is greater than 12/Re, where Re is the Reynolds Number (Chow, 1959). The Reynolds Number (Re) is defined as VL/v, where V is velocity in fps, L is characteristic length in feet, and v is the kinematic viscosity. L can be assumed as flow depth for a wide-open channel. These standing waves can cause flow to exceed freeboard limits and possibly damage the channel lining. The design engineer may resolve pulsating flow issues either by adjusting the channel slope to prevent the development of these waves or providing additional freeboard to account for the height of the standing wave. It is important to estimate roll wave height as part of freeboard design for supercritical channels. Detailed discussions and design procedures and examples regarding roll wave height can be found in LACFCD (1982) and Brock (1967).

Theoretically, slug flow will not occur when the Froude Number is less than two ($FR<2.0$). To avoid slug flow when the Froude Number is greater than 2.0, the channel slope shall be as follows:

$$S \leq \frac{12}{RE} \tag{8.3}$$

where:

S	=	channel slope (ft/ft)
RE	=	Reynolds Number, $RE = \frac{uR}{v}$ (dimensionless)
u	=	mean design velocity (ft/s)
R	=	hydraulic radius (ft)
v	=	kinematic viscosity of water (ft^2/s)

More detailed technical discussion about roll wave development and propagation is beyond the scope of this manual. Several references, including Chow (1959) and Clark County (1999) provide further discussion of this topic. The Los Angeles County Flood Control District (1982) has developed nomographs that provide freeboard allowance for roll wave height based on empirical research at the California Institute of Technology (Brock, 1967). The District will accept use of these nomographs.

8.3.9 Superelevation

Superelevation is a function of flow velocity, channel geometry, and channel alignment. Horizontal curves in a channel cause the maximum flow velocity to shift toward the outside of the bend. The rise in the water surface along the outside of the bend is referred to as superelevation.

8.3.9.1 Calculating Superelevation

Superelevation shall be calculated based on the water surface profile developed pursuant to

Section 4.4, including:

- Use of the maximum Manning's roughness coefficient expected during the lifetime of the channel, as described in Table 8.1 for the 'Capacity' design criteria.
- Full consideration of gradually varied flow profiles and rapidly varied flow impacts (jumps, piers, junctions etc.).

The following equation is recommended to estimate the magnitude of the superelevation:

$$\Delta y = \frac{CV^2 T_W}{g r_c} \tag{8.4}$$

where:

Δy = rise in water surface between water surface elevation at channel centerline and water surface elevation at outside of bend (ft)

C = curvature coefficient

Condition	Curvature Coefficient
Subcritical – Rectangular Channel	0.5
Subcritical – Trapezoidal Channel	0.6
Supercritical – Rectangular Channel	1.0
Supercritical – Trapezoidal Channel	1.3

r_c = radius of curvature at channel centerline (ft)

T_W = water surface top width for the design water depth at channel centerline (ft)

V = average channel velocity (ft/s)

g = gravitational acceleration (ft/s^2)

Designers are cautioned to avoid curves in channels with supercritical flows, particularly for trapezoidal channels, wherever possible. The shift in the velocity distribution may cause cross-waves to form, which may persist downstream and could severely limit the hydraulic capacity of the channel.

8.3.9.2 Minimizing Superelevation

In order to minimize superelevation, the recommended design practice is to design the channel radius of curvature to limit the superelevation of the water surface to 2 feet or less. This can be accomplished by assuming Δy in Equation (7.29) is equal to 2 feet (or the desired superelevation value) and then solving for r_c. If a superelevation of 2 feet or less cannot be accomplished, the designer may consider the installation of center pier(s). Piers generally will split the flow, cause a slight increase in velocity while significantly lowering the top width of flow (by effectively dividing it into two channels) and therefore reducing the superelevated water surface elevations on each side of the pier.

8.3.9.3 Length of Superelevation

When determining superelevated water surfaces for freeboard without easement curves, begin the surface change at a point 5 L' downstream of the B.C. of curve with no superelevation, taper to maximum superelevation at a point 3 L' downstream of the B.C. of curve, carry maximum superelevation to the E.C. of curve, and taper to no superelevation at a point 2 L' upstream of the E.C.

$$L' = \frac{T}{\tan\beta} \tag{8.5}$$

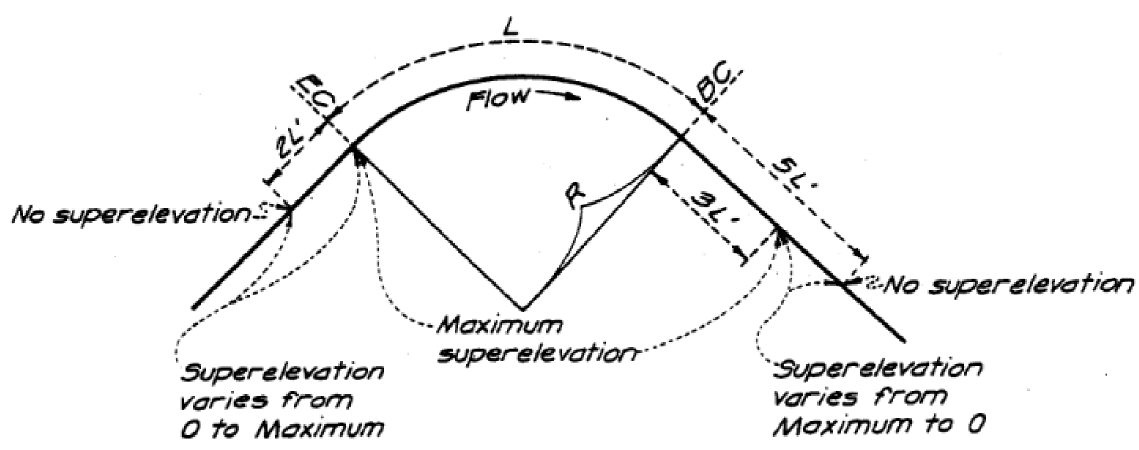
where:

T = top width (ft)
= b for rectangular channels
= b + 2 ZD for trapezoidal channels

β = wave front angle
= $\sin^{-1}\frac{gD}{V} = \frac{1}{F}$

F = Froude Number (Equation 4.2)

Figure 8.8: Superelevation Allowance



8.3.9.4 Additional Requirement for Curves on Supercritical Trapezoidal Channels

Additional 'easement curves' shall be constructed on both ends of all supercritical trapezoidal channel curves, as described in Section C-3.2 of the LA County Hydraulics Manual. Easement curves are alignment transition curves, employed upstream and downstream of circular curves, when supercritical flow exists in open channels.

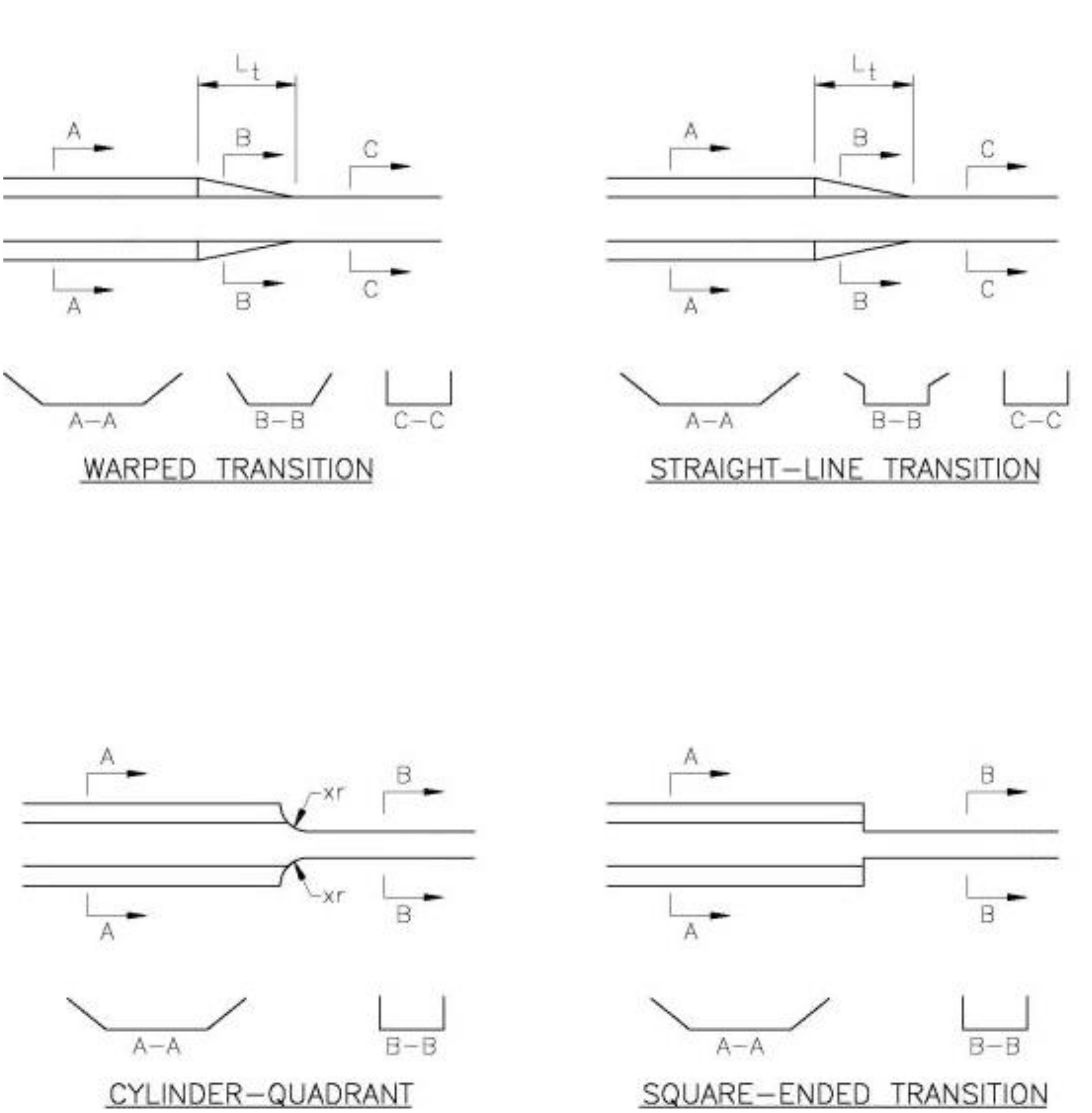
8.3.10 Transitions

Flow transitions occur whenever there is a change in channel size or shape. Properly designed flow transitions should mimic the expansion or contraction of natural flow boundaries to the extent possible and should minimize surface disturbances from cross-waves and turbulence. Drop structures are special transitions designed to dissipate excess energy over a short distance creating what is known as a hydraulic jump. Subcritical transitions shall satisfy the minimum transition lengths described in Section 8.3.10.2. Supercritical transitions shall satisfy the minimum transition lengths described in Section 8.3.10.3.

8.3.10.1 Types of Transitions

The District recommends using the warped transition which is the most efficient. See District Standard Drawing CH329. See Figure 8.9 for typical transition structure types.

Figure 8.9: Transition Structures



8.3.10.2 Transitions – Subcritical Flow

Subcritical flow transitions occur when transitioning between one subcritical channel section to another subcritical channel section (expansion or contraction).

Subcritical Transitions – Contractions

The energy loss created by a contracting section may be calculated using the following equation:

$$H_t = K_{tc}\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \quad (8.6)$$

where:

H_t = energy loss (ft)
K_{tc} = contraction transition coefficient (0.10 for warped transition)
V_1 = upstream velocity (ft/s)
V_2 = downstream velocity (ft/s)
g = gravitational acceleration (ft/s^2)

Subcritical Transitions – Expansions

The energy loss created by an expanding transition section may be calculated using the following equation:

$$H_t = K_{te}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad (8.7)$$

Expansion loss coefficient (K_{te}) is 0.20 for warped transition sections.

Subcritical Transition Length

The length of the transition section shall be long enough to keep the streamlines smooth and nearly parallel throughout the contracting section. Experimental data and performance of existing structures have been used to estimate the minimum transition length per Equation (8.8).

$$L_t \geq 0.5\, L_c(\Delta T_w) \quad (8.8)$$

where:

L_t = minimum transition length (feet)
L_C = length coefficient (dimensionless)
ΔT_w = difference in the top width of the normal water surface (not influenced by transition) upstream and downstream of the transition (feet)

Table 8.6 summarizes the transition length coefficients for subcritical flow conditions. These transition length guidelines are not applicable to cylinder-quadrant or square-ended transitions. For flow approach velocities of 12 fps or less, the transition length coefficient (L_c) shall be 4.5. This represents a 4.5L:1W expansion or contraction, or about a 12.5-degree divergence from the channel centerline. For flow approach velocities of more than 12 ft/sec, the transition length coefficient (L_c) shall be 10. This represents a 10L:1W expansion or contraction, or about a 5.75-degree divergence from the channel centerline.

Table 8.6: Transition Length Coefficients for Subcritical Open Channels

Flow Approach Velocity (V) (ft/s)	Transition Length Coefficient (L_c)
≤12	4.5
>12	10

8.3.10.3 Transitions – Supercritical Flow

The design of a supercritical flow transitions is more complicated than a subcritical transition due to the potential damaging effects of an oblique hydraulic jump occurring at an angle to the direction of flow created by the transition. The oblique jump can result in cross waves and high flow depths that can cause damage if not properly accounted for in the design. Supercritical transitions can be avoided by designing the channel so that a hydraulic jump occurs upstream of the transition. Supercritical flow contraction transitions are strongly discouraged and require special exceptions by the District for use in District facilities. For additional guidance, see Army Corps of Engineers – Design of Flood Control Channels (EM 1110-2-1601), pages 2-8 through 2-10.

8.3.11 Maintenance Access and Safety

The District recommends the following maintenance access features to be considered for all open channels. For District-maintained facilities the following maintenance access and safety features are required.

8.3.11.1 Maintenance Access Roads

Any right of way encompassing a District maintained channel shall be wide enough to provide for the channel structure and adequate maintenance access. Right of way/Easements shall be placed on one side of a lot or ownership lines in new developments as well as in existing development where conditions permit.

- Channels with a top width of less than 20 feet require a minimum 20-foot-wide maintenance access road parallel and adjacent to **one side** of the channel. An additional 1.5 feet (minimum) shall be provided from the edge of the channel structure on the non-access road side to the right of way limit.
- Channels 20 feet or more in top width require a minimum 20-foot-wide maintenance access road on **both sides** of the channel. An additional 1.5 feet shall be provided from the edge of the access roads to the right of way limits.

Maintenance access road(s) adjacent to channels shall be configured as shown in District Standard Drawings CH323, CH324, CH325, and CH326. Access ramps to channel inverts are shown in District Standard Drawings CH330, CH331, CH334, and CH335. In some cases (e.g., a long reach excavator is required for maintenance), a wider access road may be required for specialized equipment to reorient and perform intended task.

Minimum inside turning radius of maintenance access roads is 50'.

8.3.11.2 Invert Access Ramps and Turnarounds

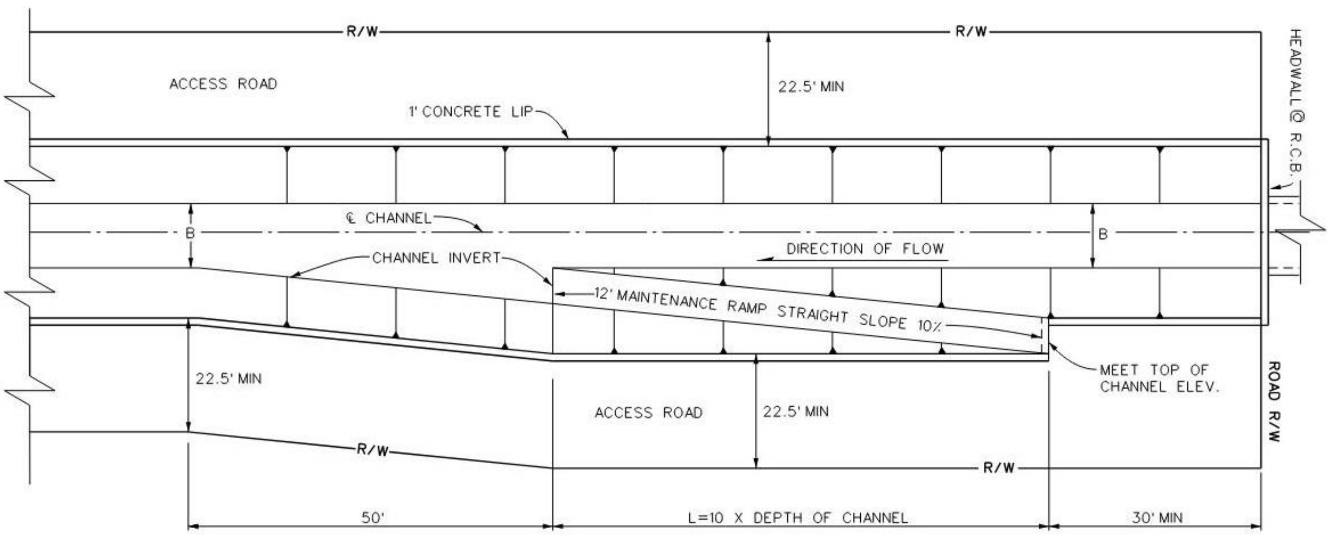
Vehicular access to the invert of District maintained drainage and flood control channels must be provided at periodic intervals to permit the efficient removal of sediment and accumulated debris and to facilitate structural maintenance. Figure 8.10 illustrates a typical ramp design in a trapezoidal channel based on RCFC Standard Drawing CH330.

Access ramps must be at least 12 feet wide, with a maximum grade of ten percent (10%) per District Standard Drawings CH330, CH331, CH334 and CH335. Generally, invert access ramps should be provided at intervals of 1,000 feet or less, and at the upstream and downstream side of every culvert and street crossing where maintenance access through the culvert itself is not an acceptable alternative. Access over or around drop structures also needs to be considered.

Vehicular turn around areas shall be provided per District Standard Drawing M827.

Channel bottom widths should be designed in consideration of maintenance requirements for the channel lining and are to be no narrower than described in Section 8.3.6 unless otherwise approved by the governing agency.

Figure 8.10: Traverse Slope Channel Access Right Angle Channel Access



Recommendations for access ramps are summarized as follows:

1. For subcritical flow, the hydraulic consequences of occasional access structures are minor. For supercritical flow, the hydraulic consequences of channel cross sectional changes can be major. Hydraulic jumps or oblique waves can jeopardize the entire channel.
2. When used in supercritical flow areas, ramps should be oriented downstream.
3. The Froude number approaching a ramp directed downstream should generally not exceed 2.2 when the ramp is located on one bank only and 2.4 when two ramps are located on both banks across from each other.
4. Depending on the depth of flow in the channel, at some point the water depth will overtop the ramp and expand to occupy the ramp width, then at the downstream end will contract again to match the downstream channel width. The hydraulic effect of this dynamic can be calculated using interim cross sections that represent the geometry at various points along the slope of the ramp, as well as a transition structure at the downstream end. The hydraulics across this system can be calculated and the transition structure can be determined using the methods described in Chapter 8 . For access ramps on supercritical channels, the transition at the downstream end of the ramp will function as supercritical contraction and should be designed to meet the requirements for such structure as described in Chapter 8 .

8.3.11.3 Safety

Concrete, shotcrete, or smooth sided soil cement channels should consider emergency escape ladders (District Standard Drawing MH259) or equivalent.

Fencing or access barriers shall be located a minimum of 6 inches inside the right of way boundary lines unless otherwise approved.

Deep channels, steep side-slopes, and high flow velocities all can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer must always consider the safety aspects of any design. Specific safety requirements shall be determined on a case-by-case basis in consultation with the governing agency. Please refer to Chapter 1 , Section 1.3.5 of this manual for additional discussion regarding safety.

Table 8.7: Channel Maintenance Requirements

Channel Type	Maintenance Requirements
Concrete Sides and bottom	Maintenance required for full channel, access road width, and 1.5 feet outside access road (See R/W limits per Standard Drawing CH326)
Riprap Sides and bottom	Maintenance required for full channel, access road width, and 1.5 feet outside access road (See R/W limits per Standard Drawing CH325)
Concrete or riprap sides and soft bottom channel	Maintenance required for full channel, access road width, and 1.5 feet outside access road (See R/W limits per Standard Drawing CH323 & CH325)
Concrete, riprap, or soil cement bank protection only (invert unmaintained)	Maintenance limit measured from outside of access road extending toward the channel centerline to bottom of toe down, over 12 feet toward channel centerline, then up at 1:1 slope to channel bottom (See Detail A on Standard Drawing CH323 & CH325)
Earthen channel	Maintenance required for full channel, access road width, and 1.5 feet outside access road (See R/W limits per Standard Drawing CH324)
Earthen channel with habitat mitigation area in channel bottom	Maintenance limit measured from outside of access road extending toward the channel centerline to toe of channel slope, then over 15 feet toward centerline

8.3.12 Confluence Junction

The hydraulic design criteria for confluence junctions between a main channel and side channel should be based on Section 4-4 in USACE (1994). One of the key design parameters is that the junction angle between the main and tributary channel alignment should be as slight as practical given the site constraints for subcritical flow conditions and not greater than 12 degrees for supercritical flow conditions. Other design criteria and procedures are provided in USACE (1994).

Junction hydraulic calculations shall be per Chapter 4 or using approved software such as WSPGW (see Section 1.3.4).

8.3.13 Side Drainage

As discussed in Section 8.3.2, engineered channels are generally built along natural topographic low points. This means that the engineer must identify and plan for how drainage from adjacent land will safely enter the channel. Uncontrolled over-side drainage is not allowed as it may damage the channel or channel lining.

Typically such flows must be collected in swales along the outside of the maintenance road (may require additional right of way), and delivered to the channel in one of two ways:

- A catch basin or drop inlet that will convey the water into the channel via an underground pipe.
- A constructed drainage apron per District Standard Drawing CH333.

8.3.14 Environmental Permitting

Open channel facilities are often located within or adjacent to sensitive environmental areas. The design engineer must identify which permits may be necessary from various agencies, including but not limited to: U.S. Army Corps of Engineers (e.g., Section 404 Wetland Permit), California Department of Fish and Wildlife (e.g., Section 1600 Permit), and California State Water Resources Control Board (e.g., Section 401 Water Quality Certification). It is important that the final permits and/or permit conditions allow not only for the construction of the facility, but also allow for the perpetual maintenance of the facility without future permitting or mitigation requirements.

8.3.15 No Mitigation Areas within District Drainage Facilities

Natural conveyance areas shall avoid including environmental mitigation areas (e.g., riparian habitat) to facilitate proper maintenance and secure hydraulic capacity. The District cannot have any limitations on conducting the maintenance required to ensure the integrity and hydraulic capacity of the drainage facilities. Therefore, environmental mitigation areas will not be allowed within any drainage facility that is intended to be transferred to the District. This includes, but is not limited to, mitigation no touch zones, mitigation vegetation strips, and conservation easements or areas. Other mitigation alternatives exist for a project proponent, such as offsite mitigation or purchasing credits from a mitigation bank.

8.3.16 Boundary Conditions

The starting water surface elevation for hydraulic grade line computations should be based on FEMA requirements (FEMA, 2003). In general, the starting water surface elevation for computations should be based on a known water surface elevation. In cases where the starting water surface elevation isn't known, an assumed water surface elevation based on normal depth (or slope-area) may be utilized. When starting with an assumed water surface elevation, the model should be started far enough downstream from the area of interest that the starting water surface elevation has minimal impact on the water surface elevation at the area of interest. When modeling a tributary to a larger watercourse, it may not be prudent to utilize the maximum water surface elevation in the larger watercourse as the starting water surface elevation for the tributary, as the two watercourses may not have coincident peaks with respect to time, or the maximum water surface elevation in the tributary may actually be higher than that in the larger watercourse. In the case where peaks do not coincide, normal depth may be utilized as the tributary starting water surface elevation. Coincident peaks should be assumed if a) the ratio of the drainage areas for the two watersheds lies between 0.6 and 1.4, b) the times of peak flows are similar for the two combining watersheds, or c) the likelihood of both watersheds being covered by the same storm being modeled is high. If gauge records are available for the basin, guidance for coincidence of peak flows should be taken from them.

8.3.17 Debris Modeling

The hydraulic modeler shall estimate blockage due to debris at bridge piers and RCB walls based on factors such as the anticipated debris load from upstream watershed (urbanized watersheds will have less debris load than natural watersheds) and pier geometry (large diameter piers will accumulate less debris than small diameter piers). Additionally, where mitigation measures are utilized, such as pier nose extensions (debris fins), modeling blockage from debris may not be necessary. Where debris is anticipated, bridge piers and RCB walls are to be modeled with conveyance blocked on each side of the obstruction a distance agreed to by the local governing agency. As a minimum, use one foot on each side of the pier, however, a greater value may be warranted based on site conditions.

8.3.18 Channel Stabilization Design Criteria

Design of new channels shall require analysis that addresses sediment transport, scour, lateral migration, and river mechanics. Calculations are required to demonstrate that the type of bank protection (riprap, concrete, etc.) is suitably sized to resist hydraulic forces (tractive shear, impingement, buoyancy, etc.) at the design frequency peak flow. The minimum factor of safety applied to hydraulic forces on structural components shall be 1.5 based on 100-year frequency peak flow.

Consideration shall be given to how the upstream and downstream floodplain conditions will impact the proposed channel. The effects of existing and potential mining and fill operations shall be addressed. Overbank flooding upstream of the channelization shall be analyzed to demonstrate that design flows enter and are contained within the improved channel. The design and analysis shall address the potential impacts of known future modifications proposed by others. A stable transition of new channelization to the existing floodplain/floodway is required at both the upstream and downstream ends.

8.4 DESIGN GUIDELINES – NATURAL WATERCOURSES

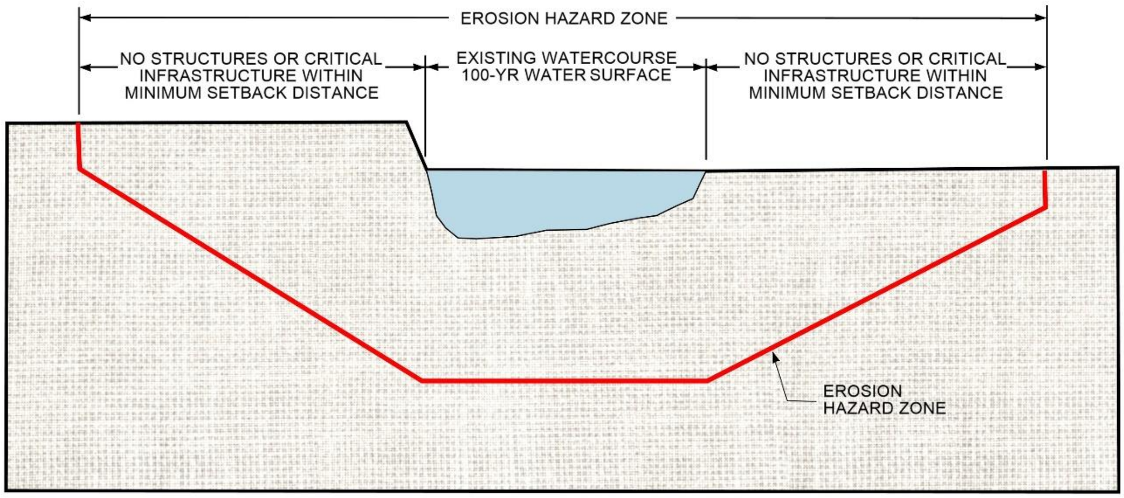
8.4.1.1 Erosion Hazard Setback

For development adjacent to natural watercourses, an erosion hazard setback is required to protect against erosion due to lateral migration, see Figure 8.11. An erosion hazard zone can be defined as an area where erosion may potentially result in damage to a resource. A "resource" may be inclusive of private or public houses, buildings, apartments, fences, utilities, infrastructure, or other feature of appreciable value. The recommended erosion hazard setback shall be as follows:

- For watercourses conveying a 100-year flowrate greater than 1,000 cfs (including major watercourses such as Santa Ana River, Murrieta Creek, Whitewater, etc.), the setback shall consider both the watercourse 100-year floodplain extents and an analysis of long-term scour and lateral migration that considers anticipated changes to upstream sediment supply. The particular aspects of these detailed studies shall be discussed with and agreed to by the governing agency on a case-by-case basis prior to performing the studies. These detailed studies are beyond the scope of this manual. Reference material regarding determining channel morphology can be found in FHWA HEC-20 and Austin, TX Erosion Hazard criteria.
- For minor watercourses, with a 100-year flowrate between 500 cfs and 1,000cfs, the setback shall be a minimum of 50 feet from the limits of the 100-year water surface elevation.
- For minor watercourses, with a 100-year flowrate less than 500cfs, the setback shall be a minimum of 25 feet from the limits of the 100-year water surface elevation.

If erosion hazard setback criteria are not observed, an engineered bank protection maintained by a public entity may be required. See Section 8.9 for design criteria regarding soft bottom, bank lined channels.

Figure 8.11: Erosion Hazard Setback Zone



8.4.1.2 Base Flood Elevation Determination

Methods acceptable to FEMA for determining a Base Flood Elevation are beyond the scope of this manual, however, it is noted that where supercritical flows exist in a natural watercourse, FEMA's guidance for development in Zone A floodplains requires that the flood limits be based on a depth no less than critical depth.

8.4.1.3 Stream Restoration

Stream restoration/rehabilitation of natural channels is beyond the scope of this manual. Reference material regarding stream restorations is available from National Resource Conservation Service National Engineering Handbook Part 654 (NRCS, 2007) and South Orange County Water Quality Improvement Plan Appendix I (OCC, 2020). The District typically does not provide maintenance of natural watercourses.

8.5 DESIGN GUIDELINES – EARTHEN AND VEGETATED CHANNELS

This Section presents minimum design criteria for earthen, naturally vegetated or landscaped open channels based on District Standard Drawing CH324. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 8.3, and any special considerations that may be unique to their project.

8.5.1 Flow Regime

Earthen and vegetated engineered channels cannot be used in cases where supercritical flows exist. If supercritical flows exist for short reaches of a channel (but the rest is subcritical), those supercritical reaches can be armored consistent with the requirements of Section 8.9. The upstream and downstream end of the armored supercritical reach must include a properly designed cutoff wall.

8.5.2 Longitudinal Channel Slope

The longitudinal slope of earthen and vegetated channels shall be such that maximum permissible velocity requirements are satisfied based on the soil types and the amount/type of vegetative cover. Table 8.1 describes the n-value criteria for determining the peak velocity in the channel. Table 8.2 summarizes the maximum permissible velocity for typical earthen and vegetated channel conditions. Where conditions warrant, drop structures may be used to lower the slope and design velocities to acceptable ranges (see Section 9.2).

8.5.3 Bend Protection

The potential for scour increases along the outside bank of a channel bend due to the higher velocities and angle of attack of flows along the outside of the bend. Therefore, it is necessary to provide enhanced erosion protection in areas with erosive soils even if none is required along the straight reaches of the channel.

This bend protection can be accomplished using riprap (loose or grouted) or concrete, by following the criteria within the applicable Design Guideline within this manual. Other materials may also be feasible for use in bend protection scenarios but appropriate methods for those materials are beyond the scope of this manual. Alternative materials will require approval of the governing agency.

When scour protection is provided, channels may have a minimum radius equivalent to 1.2 times the 100-year flow top width, but in no case shall the radius of curvature be less than 50 feet.

Scour protection shall extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

8.5.4 Scour, Degradation, and Aggradation

Scour analysis, long term degradation or aggradation, and stable bed analysis should be considered when designing earthen and vegetated channels but are beyond the scope of this manual.

8.6 DESIGN GUIDELINES – CONCRETE LINED CHANNELS

Reinforced concrete is a rigid lining material that can be used for channels with limited right of way and/or high velocity flow. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 8.3, and any special considerations that may be unique to their project.

While inherently highly resistant to normal scouring conditions, concrete lined channels are susceptible to failure from: 1) cracking due to settlement of the sub-grade; 2) cracking due to the removal of bed and bank subgrade material by seepage forces; 3) cracking due to uplift from hydrostatic pressure caused by high groundwater; and 4) abrasion due to significant velocities and sediment loads within the flow.

8.6.1 Standard Drawings and Geometric Considerations

All concrete channels that will be owned and maintained by the District shall be designed in accordance with this manual as well as the applicable District Standard Drawings, including CH326 through CH332.

Lateral storm drain pipelines joining the channel (at a junction) must not protrude into the channel flow area. For channels carrying supercritical flow, there shall be no reduction in cross section area at bridges or culverts, or any obstructions in flow path.

8.6.2 Longitudinal Channel Slope

Concrete lined channels can accommodate supercritical flow conditions and, therefore, can be constructed to almost any naturally occurring slope. However, when designing a concrete lined channel with supercritical flow, the designer must use the utmost care and consider all relevant factors and risks.

The longitudinal slope of concrete-lined channels shall be such that maximum permissible velocity requirements are satisfied. Table 8.1 describes the n-value criteria for determining the peak velocity in the channel. Table 8.2 summarizes the maximum permissible velocity.

Where channels are designed for supercritical flow, great care must be taken to restrict public access to the channel. High velocities, even at shallow depths, can knock a person over increasing the drowning risk inherent with open waterways. It is generally unwise to have any curvature in a supercritical channel since they may create oscillatory waves that could extend the entire length of the channel. High velocity flow can enter cracks or joints and create uplift forces by the conversion of velocity head to pressure head causing damage to the channel lining. These risks can be minimized by using as mild a slope as possible for the system, potentially using drop structures to minimize slope.

Conversely, on very mild slopes where sediment is allowed to accumulate, vegetation may grow and increase the flow resistance. This results in a reduction in channel capacity that can cause the channel banks to overtop at less than the design discharges and, consequently, erode the overbank material leading to failure of the concrete lining.

8.6.3 Concrete Lining Structural Considerations

Method and Loads - Concrete lined channels should be reinforced at minimum for temperature variations, and for structural loads when constructed with side slopes steeper than 1.5H:1V. Structurally, the more vertical the side slope becomes the more it will act as a retaining wall and must be designed accordingly. Structural calculations and design must be performed in accordance with the current version of ACI 318, ACI 350 and USACE EM 1110-2-2104 - Strength Design for Reinforced Concrete Hydraulic Structures. USACE EM 1110-2-2502 - Retaining and Flood Walls provides detailed methodologies for determining the magnitude and distribution of different loads on retaining walls. At minimum, the geotechnical, hydraulic, and vehicle loading scenarios summarized in Figure 8.12 and Figure 8.13 must be analyzed. Other combinations may be required based on site specific conditions.

Figure 8.12: Minimum Loading Conditions for Rectangular Channels. Based on EM 1110-2-2007.

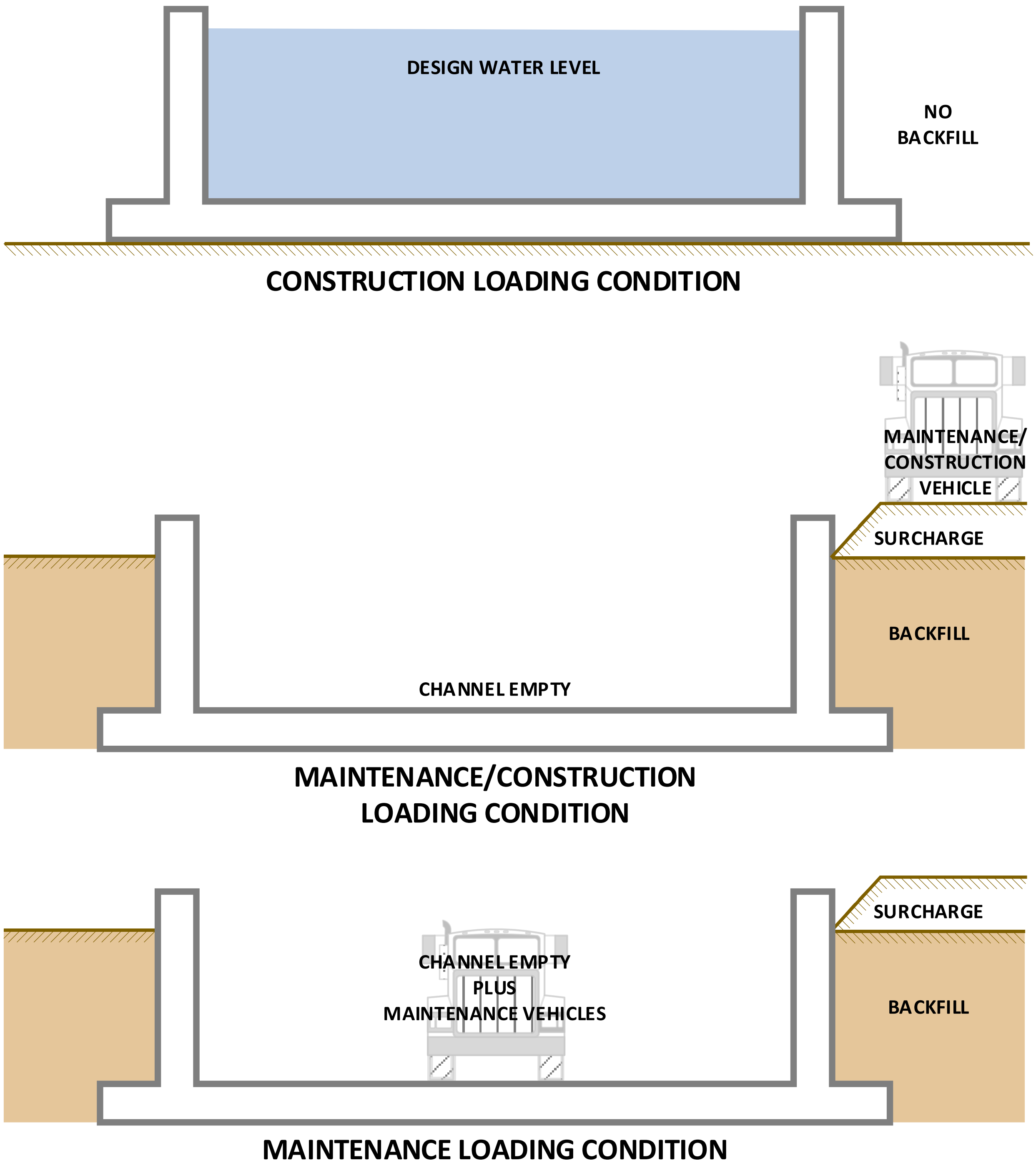


Figure 8.13: Minimum Loading Conditions for Rectangular Channels. Based on EM 1110-2-2007.

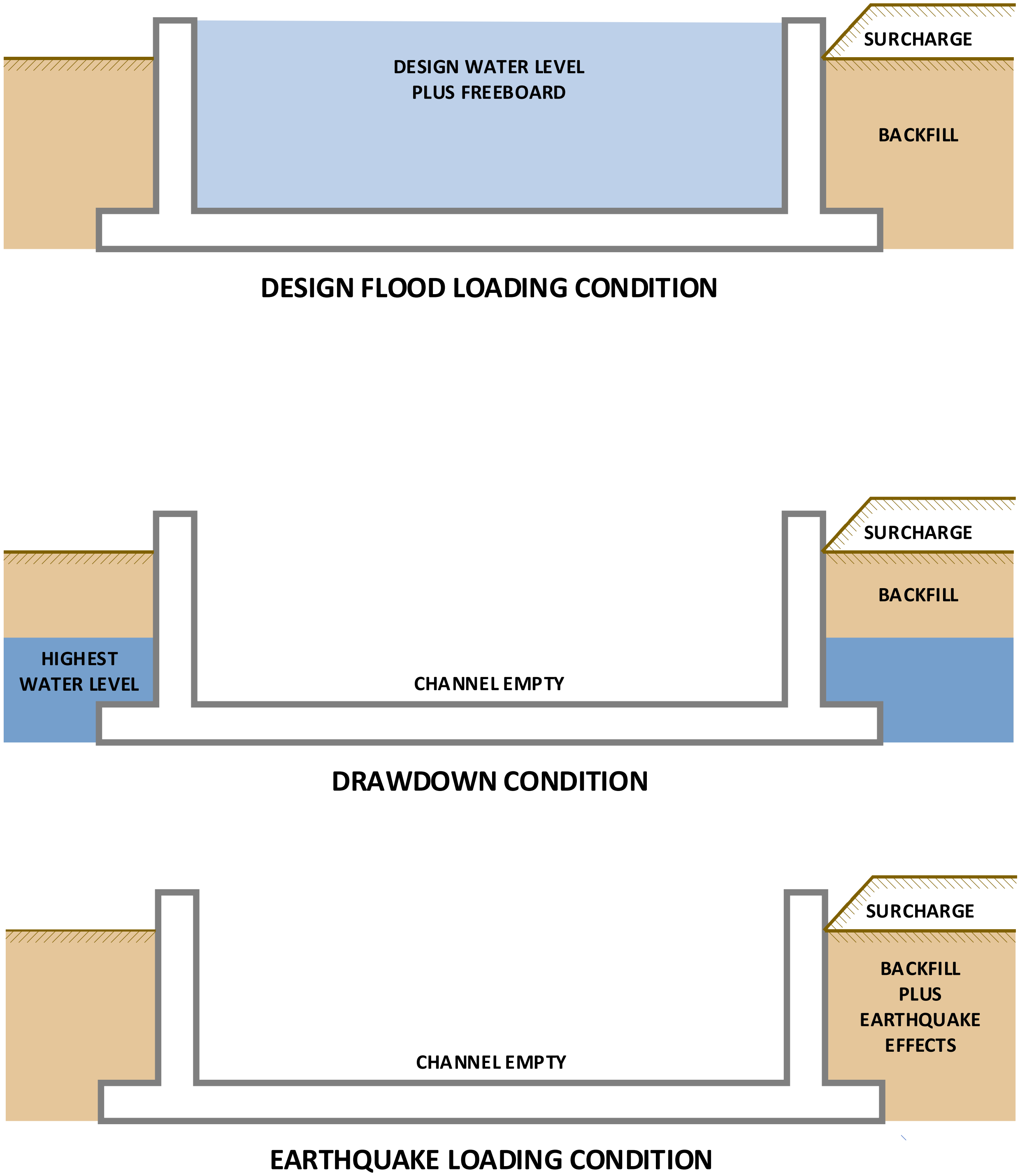


Table 8.8: Loading Types. Based on Chapter 3 of USACE EM 1110-2-2104

Load Category	Load Types
Permanent Loads	• Dead Loads (Includes weight of permanent structural features) • Vertical Earth • Lateral Earth • Surcharge
Temporary (intermittent static) Loads	• Peak Hydrostatic – Flood • Construction/Maintenance Live Loads (Assume HL-93)
Dynamic (impulse) Loads	• Earthquake

The Load and Resistance Factor Design (LRFD) shall be used for both structural and geotechnical evaluations for the design of reinforced concrete-lined channels. The load factors for different load combinations for reinforced concrete-lined channels and retaining walls shall be based on those indicated in Chapter 3 of USACE EM 1110-2-2104 - Strength Design for Reinforced Concrete Hydraulic Structures. The LRFD strength reduction and overload factors shall match those in the corresponding methods from ACI 318.

Vertical soil pressures on rigid U-frame channels shall be computed assuming the invert slab as a beam on elastic foundation in accordance with theories outlined in: Heteny, M., Beams on Elastic Foundations, University of Michigan Press, Ann Arbor, 1946.

Rectangular channels should be design as "U" rigid frames, except for channels widths greater than 30 feet. Rectangular channels greater than 30-inch width may be designed as "L" walls with a connecting floater slab. Rectangular channels should also be checked for stability, upheave, soil reaction, and sliding.

Generally, if side slopes steeper than 2:1 are used, then safety and structural requirements become a greater concern. Steeper side slopes are a safety concern due to the difficulty in walking the slope and being a falling hazard for maintenance crews as well as unauthorized members of the public should they gain access to the channel. Structurally, the more vertical the side slope becomes the more it will act as a retaining wall and must be designed accordingly. Design of the lining must also include consideration of anticipated vehicular loading from maintenance equipment.

Scour and Abrasion Considerations - Concrete lined channels are usually designed for high velocity flow conditions. In cases where a concrete channel is expected to carry a large debris load or abrasive sediment material at high velocities, it shall have a thickened lining section and/or other measures, such as increased aggregate size, to help extend the design life for the facility. Concrete lining shall have a minimum thickness of 6 inches for flow velocities less than 30 fps. For flow velocities over 30 fps, concrete lining shall have a minimum thickness of 8 inches in order to provide a minimum of 3.5 inches of cover over reinforcing steel.

Joints - Joints in the lining must be designed in accordance with standard structural analysis procedures considering the size of the channel, thickness of the lining, and anticipated construction techniques. Typically, joints are installed to control cracking both for temperature and constructability considerations and where discrete structures are joined. Some must have continuous reinforcement extending through construction joints, while others the steel is not continuous through the joint and must be terminated a minimum distance from the joint. Required joints are described on District Standard Drawings.

Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major debris blockage.

The concrete lining must be keyed into the adjacent over-banks with a longitudinal cut off wall as shown in District Standard Drawing CH323 and CH326.

Subgrade Considerations - Long-term stability of concrete lined channels depends in part on proper subgrade conditions. Undisturbed soils may be satisfactory for a foundation for lining without further treatment, but this must be confirmed by a geotechnical evaluation. Often the channel bed and/or banks may require over-excavation and recompaction, or other treatments. Expansive clays are usually an extreme hazard to concrete lining and should be avoided or mitigated in consultation with a geotechnical engineer.

Transitions - Since concrete-lined channels are often used at locations where smaller channel cross sections are required, transitions to match the existing watercourse geometry will be required both upstream and downstream of the lined channel section. Such transitions are intended to minimize head loss and turbulence between the concrete-lined and existing watercourse. Transitions must be lined with concrete or other scour resistant material to reduce scour potential and must incorporate transverse cutoff walls where the transition meets the adjacent watercourse. The cutoff wall will help reduce seepage forces and prevent lining failure due to scour, undermining, and piping. The depth of cutoff walls should extend below the expected scour depth. Determination of expected total scour depth is beyond the scope of this manual, refer to LA County Department of Public Works Sedimentation manual for guidance and insight into sedimentation and scour.

Subdrains - The probability of damaging the concrete lining due to hydrostatic pressure and subgrade erosion can be greatly reduced by providing underdrains. Reference District Standard Drawing CH332 for subdrain layout.

8.6.4 Public and Personnel Safety

Concrete channels are generally not designed to be safe for public use or access, so great care must be taken to restrict public access to the channel. Concrete channels typically have high velocity flows which, even at shallow depths, can knock a person over increasing the drowning risk inherent with open waterways. Additionally, accessing or escaping a channel can be difficult due to the steep side slopes associated with concrete channels, causing a hazard for maintenance crews as well as unauthorized members of the public should they gain access to the channel. The following measures should be considered as part of any concrete channel design.

Fencing - At minimum the property perimeter should be fenced in such a manner to exclude the public from access to the site. Where steep slopes or vertical drops exist within a channel (such as for wingwalls, headwalls, rectangular channels), the designer should consider additional safety measures at the top of the wall, such as a three-wire cable fence per Caltrans Standard B11-47.

Ladders - The designer should consider installing emergency escape ladders/steps (District Standard Drawing MH259) or similar within concrete channels at appropriate points (such as bridge crossings, inlets, monitoring equipment). For supercritical channels, the emergency escape ladders may need to be recessed into the concrete as to not intrude into the flow area. While the flow obstruction of the ladder itself is ignorable, debris can catch on Standard MH259 ladder rungs which can have an inordinate impact on supercritical flows. Recessing the ladder into the wall will require special detailing on the plans.

8.7 DESIGN GUIDELINES – RIPRAP LINED CHANNELS

Riprap can be an effective channel lining material if properly designed and constructed. The choice of riprap over concrete or other hardened linings usually depends on the availability of rock with suitable material properties and at a cost that is competitive with alternative lining systems. Riprap linings can also be used to increase the n-value and, therefore, reduce velocities of flow. Riprap transitions and bends in otherwise non-riprap channels are also considered riprap-lined channels and shall be designed in accordance with the design standards outlined in this Section. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 8.3, and any special considerations that may be unique to their project.

8.7.1 Standard Drawings and Geometric Considerations

All riprap lined channels that will be owned and maintained by the District must be designed in accordance with this manual as well as the applicable District Standard Drawings, including, but not limited to, CH325, CH332 (when grouted), and CH335.

8.7.2 Longitudinal Channel Slope

The longitudinal slope of riprap-lined channels shall be such that maximum permissible velocity requirements are satisfied for the rock size being utilized. Table 8.1 describes the n-value criteria for determining the peak velocity in the channel. Table 8.2 summarizes the maximum permissible velocity for riprap lined channels. Longitudinal slopes greater than 2% are not permitted; instead, where steeper channel slopes are needed to accommodate project constraints, drop structures are to be installed with a maximum allowable slope of 2% between drop structures (see Section 9.2). This slope limitation is consistent with FHWA Hydraulic Engineering Circular No. 23 (Section 4.2.2 Equation 4.1).

8.7.3 Horizontal Channel Alignment

Horizontal channel alignment shall be carefully coordinated with the riprap size and configuration (i.e., fully lined, or bank-lined with soft bottom) and checked to ensure adequate erosion protection at the toe of the channel bank to account for variations in flow velocity through curves.

8.7.4 Rock Riprap Material

Rock used for riprap shall be hard, durable (non-weathered), granitic rock that is fractured and angular in shape and free from cracks and organic matter. Neither the breadth nor thickness of a single stone shall be less than one-third of its length; rounded stone shall be avoided. Rock having a minimum specific gravity of 2.65 is preferred. Rock with a lower specific gravity may be approved on a case-by-case basis. Construction debris (e.g., broken concrete or asphalt) is not acceptable for use as riprap. Table 8.9 summarizes common rock riprap gradations from Caltrans and the Greenbook. Section 200 of the Greenbook and Section 72 of the Caltrans Standard Specifications provide more detailed construction information for rock riprap material that is acceptable to the District.

8.7.5 Rock Riprap Stone Gradation and Size

This section discusses the selection of rock riprap gradation for open channels.

8.7.5.1 Rock Gradation

Section 72 of the Caltrans Standard Specifications provides all construction and material specifications for Riprap designs[3]. Table 8.9 summarizes common rock riprap (or Rock Slope Protection "RSP") Class by median particle size and median particle weight, adapted from Caltrans and the Highway Design Manual Chapter 870. Other standard riprap specifications, such as from the Federal Highway Administration (FHWA) or Corps of Engineers, may also be acceptable for facility design when appropriately applied.

Table 8.9: RSP Class by Median Particle Size and Weight

Nominal RSP Class by Median Particle Diameter[a,b,c]		Nominal Median Particle Weight W_{50}	d_{15} (in)		d_{50} [d] (in)		d_{100} (in)	Placement Method [f]
Class	Diameter (inches)		Min	Max	Min	Max	Max	
I	6	20 lb	3.7	5.2	5.7	6.9	12.0	B
II	9	60 lb	5.5	7.8	8.5	10.5	18.0	B

[3] "California Bank and Shore rock Slope Protection (RSP) Design" layered design methodology and its associated gradations have become obsolete and are no longer used by the District.

Nominal RSP Class by Median Particle Diameter[a,b, c]		Nominal Median Particle Weight W50	d_{15} (in)		d_{50} [d] (in)		d_{100} (in)	Placement Method [f]
Class	Diameter (inches)		Min	Max	Min	Max	Max	
III	12	150 lb	7.3	10.5	11.5	14.0	24.0	B
IV	15	300 lb	9.2	13.0	14.5	17.5	30.0	B
V	18	1/4 ton	11.0	15.5	17.0	20.5	36.0	B
VI	21	3/8 ton	13.0	18.5	20.0	24.0	42.0	A or B
VII	24	1/2 ton	14.5	21.0	23.0	27.5	48.0	A or B
VIII	30	1 ton	18.5	26.0	28.5	34.5	48.0	A or B
IX	36	2 ton	22.0	31.5	34.0	41.5	52.8	A
X	42	3 ton	25.5	36.5	40.0	48.5	60.5	A
XI	46	4 ton	28.0	39.4	43.7	53.1	66.6	A

[a] Rock grading and quality requirements per Caltrans Standard Specification.
[b] For Rock Slope Protection (RSP), use Class 8 RSP fabric. For RSP Classes IX-XI, use Class 10 RSP fabric. RSP-fabric type geotextile and quality requirements per 96 Rock Slope Protection Fabric of the Caltrans Standard Specifications.
[c] Intermediate or B dimension (i.e., width) where A dimension is length and C dimension is thickness.
[d] d%, where % denotes the percentage of the total weight of the graded material.
[e] Values shown are based on the minimum and maximum particle diameters shown and an average specific gravity of 2.65. Weight will vary based on specific gravity of rock available for the project.
[f] Placement method per Caltrans Section 72.

8.7.5.2 Required Stone Size

Caltrans' Highway Design Manual (Chapter 870) presents a preferred method for determining the rock size.

Rock sizing shall be determined per methods in Table 8.10.

Table 8.10: Rock Sizing Methods

Application	Condition	Longitudinal Slope	Method
Loose Rock	Uniform or Gradually Varied Flow	$S_0 \leq 2\%$	Equation (8.9)
		$S_0 > 2\%$	Use Equation (8.9) and compare to the sizing developed for overtopping flows presented in HEC 23, Volume 2, Design Guidelines 5
	Rapidly Varied Flow or where save action is dominant	Any	See Section 9.2.4 and Shore Protection Methods per Caltrans Highway Design Manual Chapter 880
Grouted Rock	See Section 8.7.11		
Gabions	Not covered in this manual. Contact gabion manufacturer.		

$$d_{30} = y(S_f C_S C_V C_T)\left[\frac{V_{des}}{\sqrt{K_1(S_g-1)gy}}\right]^{2.5} \tag{8.9}$$

where:

d_{30} = Particle size for which 30% is finer by weight, (ft)
y = Local depth of flow, (ft)
S_f = Safety factor, (typically = 1.1)
C_S = Stability coefficient (for blanket thickness $1.5d_{50}$ or d_{100}, whichever is greater) = 0.30 for angular rock
C_V = Velocity distribution coefficient
= 1.0 for straight channels or the inside of bends
= $1.283 - 0.2 \log (R_C/W)$ for the outside of bends (1.0 for ($R_C/W>26$)
= 1.25 downstream from concrete channels
= 1.25 at the end of dikes
C_T = Blanket thickness coefficient = 1.0
S_g = Specific gravity of stone (2.5 minimum)
g = Acceleration due to gravity, 32.2 ft/s^2

V_{des} = Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment (ft/s)
d_{50} = $1.20d_{30}$ (round up to the next higher class size), (ft)

For natural channels, $V_{des} = V_{avg}(1.74\text{-}0.52\log R_c/W))$
V_{des} = V_{avg} for $R_c/W > 26$

For trapezoidal channels, $V_{des} = V_{avg}(1.71 - 0.78\log (R_c/W))$
V_{des} = Vavg for $R_c/W>8$
R_c = centerline radius of curvature of channel bend (ft)
W = width of water surface at upstream end of channel bend (ft)
V_{avg} = Channel cross-sectional average velocity (ft/s)
K_1 = Side slope correction factor
K_1 = $\sqrt{1-\left[\frac{\sin(\theta-14°)}{\sin 32°}\right]^{1.6}}$
θ = bank angle in degrees

8.7.5.3 Rock Quality Control

Quality Concerns: Hardness is of concern because the rock is subject to rough handling and impact forces. Fracturing, which leads to odd or undesirable shapes, is to be avoided. Seams or other discontinuities can lead to breakup or undesirable shapes and damage during handling. Density of the rock requires specific gravity tests.

Quality Control Measures and Inspection: A significant effort is needed in the area of rock quality control. Submittals should be required from suppliers to document quality. Rock should be durable, sound, and free of seams or fractures. The specific gravity should be a minimum of 2.40.

Specifications should include requirements for orderly procedures and appropriate equipment, both for rock and grout placement. Gradation, durability, and specific gravity tests of riprap at the quarry are needed and should only be waived for small projects where the quarry can demonstrate recent tests. Handling that results in excessive breakage should result in changed methods and/or reexamination of rock quality. Subgrades should be dewatered and stabilized. Filters and bedding layers should be reviewed for compatibility to the onsite soil conditions. Rock handling and placement is critical. Riprap should be handled selectively so that the gradation is reestablished through any given vertical section. Areas where the thickness is comprised of all materials smaller than the d_{50}, or where excessive voids or radical surface variations occur should be reworked.

Good placement techniques should result in a riprap layer with surface materials d_{50} size or greater, closely spaced with voids thoroughly chinked and locked between larger rock, top surfaces generally parallel to the plane of the overall riprap bank or surface, and no great departures in surface elevation from rock to rock.

Graded riprap should be avoided for grouting, as the smaller rock can prevent full penetration of the grout to the subgrade and can cause incomplete filling of the voids. Large rock or boulders should be placed by Caltrans Method A to minimize disturbance of the subgrade. A minimum dimension should be specified for the rock to aid field inspection. On slopes, uphill boulders should be keyed in below the tops of downhill boulders for stability. A "stairstep" arrangement where the top surface of the rock is flat and horizontal is preferable for both aesthetic and hydraulic reasons.

8.7.6 Riprap Thickness

Riprap layers must be thick enough to ensure mutual support and interlock between individual stones in each layer. The minimum riprap layer shall not be less than the diameter of the largest stone (d_{100}) nor less than 1.5 times the median stone diameter ($1.5d_{50}$). When riprap is installed underwater, the riprap layer thickness shall be increased by at least 50 percent.

The riprap layer thickness must be embedded into the bed and banks of the channel, such that the finished surface of the riprap layer results in the channel geometries required.

8.7.7 Bedding Requirements

The long-term stability of riprap linings is strongly influenced by proper bedding conditions. Rock Slope Protection fabrics are described in Caltrans Standard Specification Section 96. The RSP fabric placement ensures that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing a gravel filter of small, well graded materials.

Generally, RSP fabric should always be used unless there is a permit requirement that precludes the placement of fabric. Where RSP fabric cannot be placed, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes.

8.7.8 Channel Bend Protection

Riprap size shall be increased by one gradation through bends, unless calculations can demonstrate the stability of the straight-channel riprap gradation through the bend. For channels conveying 200 cfs or more, the minimum radius for a riprap-lined bend shall be 1.2 times the top width of design flow, and in no case be less than 50 feet. However, larger curve radius per Section 8.3.2.3 are recommended. Riprap protection shall extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

8.7.9 Transition Scour Protection

Rock-lined transitions are common when approaching culverts, bridges, or other structures. Turbulent eddies near rapid changes in channel geometry (e.g., transitions and bridges) amplify scour potential. At these locations, the riprap lining thickness shall be increased by one gradation, unless calculations can demonstrate stability of the smaller gradation through the transition section. Protection shall extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet. Section 8.3.10 contains further discussion of transitions.

8.7.10 End Treatment and Special Conditions

Upstream and downstream ends of riprap-lined channels can require particular attention from the design engineer depending on the velocities at the lined/unlined channel interface. Where adjacent unimproved channel velocities are erosive, end treatments are warranted. This is of particular concern at the downstream end of riprap lined channels where headcutting could damage the channel. End treatments may consist of concrete cut-off walls that extend down to past the scour depth or a minimum of 1-foot past the filter fabric when the anticipated scour depth is less than the riprap thickness, grouting the end 5 feet of riprap lining full thickness (only when scour is not anticipated to be greater than the lining thickness), or thickening the riprap layer to twice the normal riprap thickness being utilized for a minimum distance equal to four times the riprap thickness, where long term scour issues may be a factor. It is noted that ongoing maintenance may be expected at this interface. The Corps of Engineers' *Design of Flood Control Channels* (EM-1110-2-1601) provides specific guidance on end treatments for riprap channels. The design engineer may also consider constructing intermediate transverse cutoff walls at regular intervals to help preserve the integrity of the loose riprap channel lining.

8.7.11 Concrete-Grouted Riprap

Concrete-grouted riprap may be used when the availability of the calculated size of loose riprap is limited, or where there is a need to reduce the total thickness of an RSP revetment, or for short reaches of riprap lined channels where high velocities may preclude the use of loose riprap, such as on the sloping face of riprap grade control structures. The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions. Caltrans Highway Design Manual Chapter 870 provides a design procedure for "concreted rock slope protection" that is acceptable to the District. The District does not accept "partially concreted" or "sacked concrete" protection methods.

Size and grading of stone and concrete penetration depth are provided in Caltrans Standard Specification Section 72. Concrete used for rock grout shall meet all standards and shall be installed in accordance with procedures outlined in Greenbook Section 300-11 or Caltrans *Standard Specifications* Section 72-5.

Cutoff walls, weep holes, and underdrain requirements will need to be considered by the designer. Cutoff walls should be incorporated with transitions at both the upstream and downstream end of the concrete-grouted riprap channel to reduce seepage forces and prevent lining failure due to scour, undermining, and piping.

8.8 DESIGN GUIDELINES – SOIL CEMENT LINED CHANNELS

Soil cement has been shown to be an effective and economical method for slope protection and channel lining for projects where local soils have the right physical properties for this use. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 8.3, and any special considerations that may be unique to their project.

8.8.1 Materials

Soils that exhibit the following geotechnical characteristics are best suited for use in soil cement:

1. Between 98 percent and 100 percent pass the 2-inch (50 mm) sieve
2. Between 55 percent and 90 percent pass the No. 4 (4.75 mm) sieve
3. Between 5 percent and 35 percent pass the No. 200 (0.074 mm) sieve
4. The Plasticity Index (PI) of the fines should not exceed 12 or per soils report recommendation

If the onsite material does not meet these guidelines, the use of addition of import material may be necessary, which may make this lining approach less economical. Standard laboratory tests are available to determine the required proportions of cement and moisture to produce durable soil cement. The design of most soil cement for water control projects is based on the cement content indicated by ASTM testing procedures and increased by a suitable factor to account for direct exposure, or abrasion forces. The minimum acceptable 7-day compressive strength for soil cement is typically 450 psi, however, a greater strength may be required where high abrasive forces are anticipated.

The Portland cement shall conform to Section 90-1.02B(1&2) of the Caltrans Specifications, except Portland Cement shall be Type I, Type II or Type IV per Soils Report recommendation.

It is important that testing to establish required cement content be done with the specific cement type, soil, and water that will be used in the project.

Typically, soil cement linings are constructed by the central-plant method, where selected onsite soil materials or soils borrowed from nearby areas are mixed with Portland cement and water and transported to the site for placement and compaction.

See Table 8.2 for maximum permissible velocities for lined and unlined channels.

8.8.2 Design of Soil Cement Linings

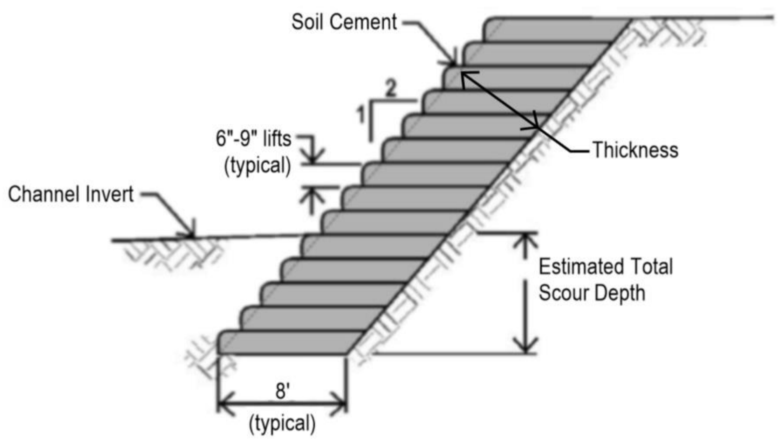
Figure 8.14 shows a composite channel consisting of an earth bottom with soil cement stabilization along the banks. On side slopes, the soil cement is constructed by placing and compacting the material in horizontal layers stair-stepped up the slope. The rounded step facing results from ordinary placement and compaction methods. Generally, an 8-foot-wide section is placed with a 2:1 sloping face in order to achieve a minimum 3.5-foot thick section as measured perpendicular to the slope. Where flatter slopes are used, a wider section is required to achieve this minimum thickness. Placement and compaction of the soil cement layers is performed using standard highway construction equipment. Figure 8.14 shows the relationship between the slope of soil cement facing, thickness of the compacted horizontal layers, the horizontal layer width, and the placement in relation to scour depth.

Soil cement channel side slope can vary from 0.5:1 to 3:1 depending on the soil type and natural angle of repose. The maximum allowable side slopes are specified in Table 8.3. Although allowed, side slopes steeper than 2:1 are not recommended due to safety issues unless right of way is a problem.

An important consideration in the design of the soil cement facing is to provide that all ends of the facing are tied into non-erodible sections or abutments. The upstream and downstream ends of the facing should terminate smoothly into the natural channel banks. A buried cutoff wall normal to the slope or other measures may be necessary to prevent undermining of the soil cement facing by flood flows.

As with any impervious channel lining system, seepage and related uplift forces should be considered and, if required, appropriate countermeasures provided, such as weep holes or subdrains. Tributary storm drain pipelines can normally be accommodated passing through the soil cement by placing and compacting the soil cement by hand methods around the drain, or by using a lean mix concrete around the drain. As with other linings of soft-bottom channels, the soil cement lining must extend to the anticipated depth of total scour below thalweg. Further design information may be found in ACI 230.1(1990), *State of The Art Report on Soil Cement*. Additional information is also available from the Portland Cement Association, Skokie, IL (http://www.cement.org/).

Figure 8.14: Soil Cement Placement Detail
(NOT TO SCALE)



8.9 DESIGN GUIDELINES – SOFT BOTTOM, BANK LINED CHANNELS

This Section presents minimum design criteria for channels where the banks are lined but the channel bottom remains either in a natural state or is graded and maintained as an earthen or vegetated channel, similar to District Standard Drawing CH323 and CH325. The design engineer is responsible for confirming that a channel design meets these criteria, the general open-channel criteria outlined in Section 8.3, and any special considerations that may be unique to their project.

8.9.1 Bank Lining Material

The lining used on the banks can be concrete, rock, or soil cement. In these cases, the bank linings should be designed in accordance with the applicable Design Guide referenced below:

Bank Lining Type	Reference
Concrete	See Section 8.6
Rock Riprap	See Section 8.7
Soil Cement	See Section 8.8

8.9.2 Longitudinal Channel Slope

For lined banks next to natural watercourses, the longitudinal slope is not modified by the project, so no particular criteria apply.

For lined banks along an engineered and maintained earthen or vegetated channel bed, the design engineer shall establish a longitudinal channel slope that maintains non-erosive velocities as described in Section 8.4. Open channels shall maintain a minimum longitudinal slope of 0.1 percent.

8.9.3 Bend Protection, Scour, Degradation and Aggradation

The issues of bend protection, scour, degradation, and aggradation discussed in Section 8.5 also apply to soft bottom channels with lined banks. See that Section for more information.

8.9.4 Roughness Coefficients

Since the banks and channel invert vary in roughness, a horizontally variable roughness coefficient (preferred) or composite roughness coefficient should be used depending on the software being used to calculate the hydraulics.

Since a composite n-value 'averages' out the roughness for the bed and banks into a single n-value, the result can underestimate the velocity on the bed of the system, and overestimate the velocity on the banks. Thus, using an approach that allows for horizontally variable roughness coefficients (such as HEC-RAS) is strongly preferred.

The appropriate roughness values for the bed and banks should be obtained from Section 8.3.4., based the criteria in Table 8.1. Generally, when considering the effects of velocity on the design of both the bed and banks (scour, rock size, etc.) the n-values associated with the 'peak velocity' criteria (in Table 8.1) should be used. When considering the capacity of the system (depth, freeboard, etc.), the n-values associated with the 'capacity' criteria (in Table 8.1) should be used.

8.9.5 Bank Toe Protection

The bank protection must extend below the channel bottom to the total scour depth for long-term stability of the lining. Total scour depth is comprised of three components: 1) long-term aggradation and degradation of the riverbed; 2) general scour due to increased velocity at contractions, or other general scour phenomenon; and 3) local scour, such as that which occurs at a bridge pier or abutment. The procedures for estimating the total scour are beyond the scope of this manual, see Evaluating Scour at Bridges (HEC-18) (FHWA, 2012) for additional information. At the discretion of the District, launchable riprap toe may be allowed based on U.S.

Army Corps of Engineers EM 1110-2-1601 or FHWA HEC-23.

8.10 DESIGN DOCUMENTATION REQUIREMENTS

See Chapter 2 for information about the drainage report and floodplain study requirements.

All channel design submittals shall include the calculated Froude Number and critical depth for each unique channel reach to identify the flow state and verify compliance with these criteria.

8.11 RECREATIONAL TRAILS

As described in Section 1.4, the District may permit the inclusion of public-use recreational trails along District-maintained facilities where such trails are maintained through a license agreement by other public entities, such as Cities and/or Parks and Recreation Districts.

8.12 SAFETY

Deep channels with steep side-slopes and high flow velocities can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer should always consider the safety aspects of open channel design. Fencing to minimize public access should be provided for all District channels regardless of depth. Concrete, shotcrete, or smooth sided soil cement channels should consider emergency escape ladders (District Standard Drawing MH259) or equivalent. In instances where open channels connect with conduits that meet the geometric and hazard requirements listed above, access barriers such as trash racks are recommended to restrict access by the general public along the entire reach of that channel. An example would be a concrete lined channel with 2(H):1(V) or steeper side slopes.

Where channel side slopes are 4(H):1(V) or flatter, and the facility is to be part of a designated trail, park, or open space area, alternative types of barriers may be considered. Designers are to consult the District, the local land-use authority, and/or responsible parks and recreation entity during the project planning process. Where open channels are located in close proximity to the road, County or City may require installation of guardrails.

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9 HYDRAULIC CONTROL STRUCTURES

9.1 USE OF STRUCTURES IN DRAINAGE

Hydraulic control structures are used in storm drainage works to control water flow characteristics such as velocity, direction, and depth. Structures may also be used to control the elevation and slope of a channel bed, as well as the general configuration, stability, and maintainability of the waterway.

Thoughtful use and integration of hydraulic control structures can reduce initial and future maintenance costs by changing the characteristics of the flow to fit the project needs, and by reducing the size and cost of related facilities.

Hydraulic control structures include channel drop structures, spillways, grade control structures, energy dissipators, and many other specific drainage works. Depending on the function to be served, the shape, size, and other features of hydraulic control structures can vary widely from project to project. Hydraulic design procedures (including model testing in some cases) that examine the structure and related drainage facilities are a key part of the final design for all structures.

This chapter is oriented toward control structures for drainage channels, outlets for storm drains and culverts, and spillways for dams. Design guidelines for other specialized conveyance measures are beyond the scope of this manual. The design professional is referred to the references cited at the end of this chapter.

9.1.1 Channel Drop Structures

Drop structures are used to reduce the effective slope of a natural or artificial channel. Typically, a drop structure extends across the entire width of the channel up to the design water surface elevation plus freeboard. When the water surface elevation exceeds the top elevation of a drop structure along the banks, flow will circumvent the structure and may cause significant erosion of the banks. Determination of what is an acceptable level of bank erosion must be considered when establishing the top of structure elevation along the banks. For additional information regarding channel drop structures, see Section 9.2.

9.1.2 Energy Dissipation Structures for Conduits

Energy dissipation structures are typically necessary at the outlets of culverts or storm drains to reduce flow velocity and to provide a transition whereby the concentrated, high velocity flow exiting the conduit is changed to a wider, shallower, and non-erosive flow. Outlet energy dissipation structures are typically rock riprap aprons, riprap, or concrete stilling basins, or cast-in-place or precast concrete structures such as impact basins. For additional information regarding energy dissipation structures, see Section 9.3.

9.1.3 Weirs and Orifices

Weirs and orifices, typically used for spillways, are conveyance features that permit controlled outflow from dams, detention, or retention basins, or laterally from channels into detention or retention basins. Engineering nomenclature for dams and basins divides these into principal spillways and emergency spillways. The principal spillway for a dam or basin is that hydraulic structure that has been designed to pass the more frequent flow events (design storm) while the hydraulic capacity of the emergency spillway is held in reserve for the rare flow events (events that exceed the design storm). An emergency spillway is designed to safely pass flows in excess of the facility design discharge in a manner that does not threaten the integrity of the principal spillway, facility embankment, or surrounding infrastructure. It also serves to pass flows normally conveyed by the principal spillway under circumstances when the principal spillway becomes plugged. Section 9.4 presents the hydraulic equations used to determine hydraulic capacity for spillways. See Chapter 10 for a more detailed discussion pertaining to how these facilities are incorporated into stormwater basins.

9.1.4 Special Channel Structures

Bridges, spur dikes, channel transitions, bifurcations, constrictions, bends, and structures for lined channels and for long conduits are examples of hydraulic structures used for special applications. Access ramps, while not a hydraulic structure, are necessary components of a channel to facilitate maintenance and can impact the hydraulics of the facility. Lastly trash racks may be utilized to prevent debris from entering the system or to prevent unauthorized access.

9.1.4.1 Bridges and Related Structures

Bridges have the potential advantage of crossing a waterway without disturbing the flow. However, for overall economic and structural reasons, encroachments and piers in the waterway are a practical reality. A bridge structure can cause significant hydraulic effects, such as an increase in the water surface elevation upstream of the bridge, and channel scour extending from upstream of the upstream bridge face to downstream of the downstream face. These conditions must be analyzed, and hydraulic structures may need to be designed to mitigate negative impacts. Spur dikes, levees, drop or grade control structures, and pier and abutment protection are types of structures designed to control hydraulic effects at bridge crossings. Refer to Chapter 7 for further discussion on bridges.

9.1.4.2 Bifurcation Structures

Bifurcations are structures that permit a portion of flow to be diverted from one storm drain facility (e.g., channel, pipe, or basin) to another facility. Similarly, side channel spillways also permit the diversion of flow. These and other types of channel junctions pose interesting design challenges, especially under supercritical flow conditions. See Section 9.5 for additional information.

9.1.4.3 Trash Racks and Access Barriers

Trash racks (e.g., Standard Plans for Public Works Construction Trash Rack Standard Plan 361-2) serve two purposes when utilized in conjunction with storm drains, culverts, and detention basin outlets. First, trash racks prevent entrapment of person(s) inadvertently swept into flood waters. Secondly, these structures prevent debris from becoming lodged in the downstream conduit. Depending upon the flow characteristics, the analysis and design considerations vary.

Access barriers may be placed at the downstream end of storm drains, culverts, and detention basin outlets to prevent the public from entering the conduit. Access barriers are typically the same configuration as trash racks, although trash racks tend to be secured rigidly top and bottom to the facility upstream headwall, whereas access barriers are typically attached only at the top of the facility downstream headwall and permitted to swivel about a horizontal axis so that the barrier can open from the force exerted by flowing water. See Section 9.7 for additional guidance.

9.1.5 Factors of Safety

Specific calculations to determine foundation stability and factors of safety against sliding, uplift, and overturning for a hydraulic structure are necessary in the design of safe structures. The factor of safety derived for a particular case depends, to a large degree, on the risk and consequence of failure. Therefore, the selected factor of safety must be appropriate for each structure being designed.

The factors of safety for sliding, uplift, and overturning all may be different for a particular structure. A general range of 1.5 to 2.0 for these factors is recommended for many types of structures subjected to a variety of loading conditions (see *Design Manual, Foundations and Earth Structures* (U.S. Navy, 1982); *Design of Small Dams* (USBR, 1987); *Design of Gravity Dams* (USBR, 1976); and *Drainage of Roadside Channels with Flexible Linings* (USDOT, 1988)).

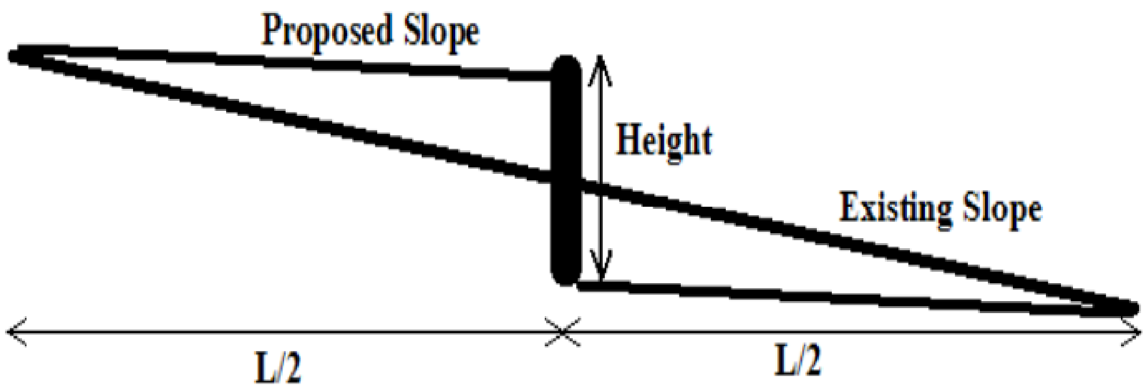
The factor of safety for trash rack clogging is discussed in Section 9.7.

9.2 CHANNEL DROP STRUCTURES

9.2.1 General

The design of stable open channels, either earthen or vegetated, or the re-stabilization of an eroding natural watercourse often requires the use of transverse channel drop and/or grade control structures to reduce the longitudinal slope to keep flow subcritical and design velocities within acceptable limits.

Figure 9.1: Drop Structure



Channel drop structures may be constructed of many materials, including concrete, riprap, grouted riprap, and sheet piles. The selection of material depends in part on hydraulic limitations, aesthetic considerations, and other site conditions such as presence of abrasive sediment bed load.

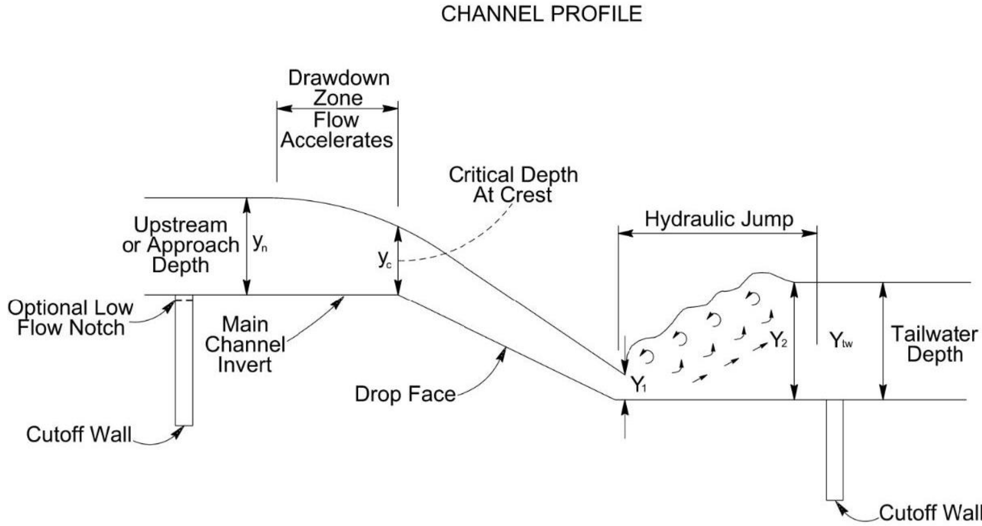
9.2.1.1 Basic Components and Function of a Drop Structure

Figure 9.2 shows a typical channel drop structure with its various components. As the slower subcritical runoff approaches the drop structure from upstream, it transitions through critical depth at the crest of the structure and becomes supercritical as water flows over the drop structure. Near the bottom of the structure a hydraulic jump is created where energy is dissipated and water transitions back to subcritical flows for the downstream channel.

The District typically does not allow stilling basins at the bottom of the drop structure unless it is designed to have positive drainage to prevent vector issues due to standing water.

The effectiveness of grade control and drop structures is dependent on many factors including flow rate, tailwater depth, and type of structure. The structures also must function over a wide range of flow rates. Therefore, it is important to confirm performance during events smaller than the maximum design flow. This may be accomplished by evaluating flows of more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions of the design flow rate (e.g., one-half, one-quarter, and further if necessary). However, normally only calculations for the full design flow are required to be submitted for review.

Figure 9.2: Typical Drop Structure Components



9.2.1.2 *Acceptable Types of Drop Structures*

The following types of drop structures are generally accepted for use in facilities that will be maintained by the District:

- Sloping Riprap Drop Structure – see Section 9.2.4
- USBR Type IX Baffled Apron – see Section 9.2.5

Table 9.1: Channel Drop Structures

Description	Upstream Flow Regime	Max. Drop Height (ft)	Max. Unit Discharge (cfs/ft of channel width)	Max. Inflow Velocity (ft/s)	Upstream Cross Section
Sloping Riprap Drop Structure	Subcritical	10	35	7	Trapezoidal
USBR Type IX Baffled Apron	Subcritical	n/a	60	n/a	Rectangular

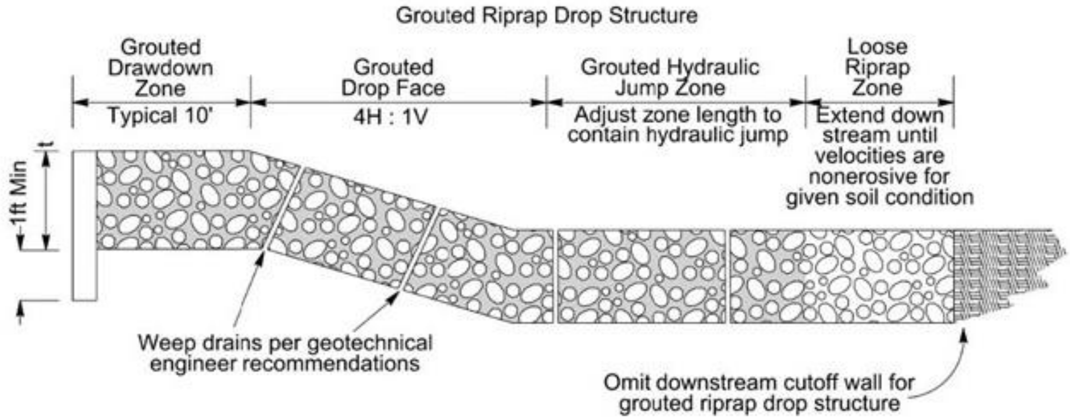
9.2.2 <u>Hydraulic Analysis</u>

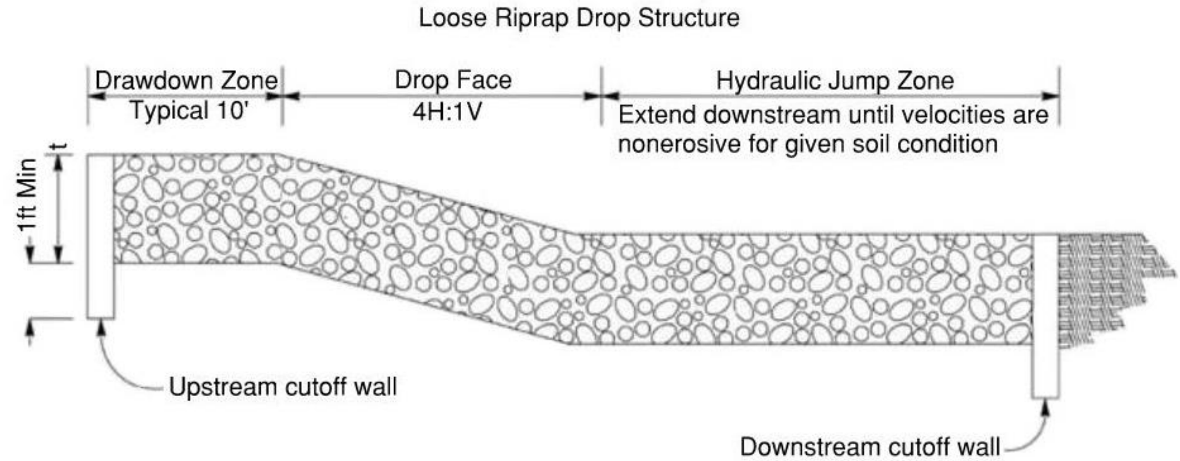
9.2.2.1 *General Procedures*

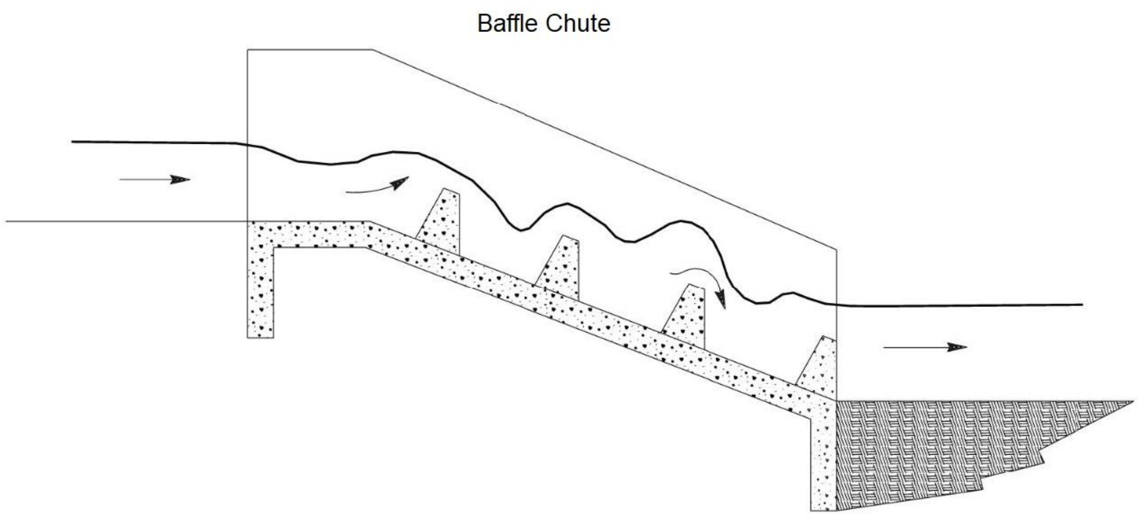
These design procedures are generalized. Use them to identify the most suitable approach, with the understanding that detailed analytical methods and design specifications may vary as a function of site conditions and hydraulic performance. A standard drop structure design approach would include at least the following steps:

1. Define the maximum design discharge (usually the 100-year) and other discharges appropriate for analysis (selected discharge(s) expected to occur on a more frequent basis, which may behave differently at the drop).
2. Select possible drop structure alternatives to be considered (Section 9.2.3).
3. Determine the required longitudinal channel slope and the total drop height required to produce the desired hydraulic conditions. The spacing of the drop structures is based on the difference in slope between the natural topographic and projected stable slope. It is critical to take care to limit the total vertical drop below any individual drop structure sills and provide adequate scour protection for the structure. Taller drop structures are expensive, significantly increase the requirement for scour protection, and increase other concerns such as safety and maintainability.
4. Conduct hydraulic analyses for the structure as described in the remainder of this Section.
5. Perform soils analyses to obtain foundation and structural design information. Use hydraulic analysis data to determine forces on the structure. Evaluate uplifting, overturning, and sliding.
6. Evaluate alternative structures in terms of their estimated capital and maintenance costs and identify comparable risks and problems for each alternative. Review alternatives with client and jurisdictional agency and maintenance entity to select final plan. (This task is not specifically a part of the hydraulic analysis criteria but is mentioned to illustrate other factors which are involved in the analysis of alternatives.)
7. Use design criteria to determine the drop structure dimensions, material requirements, and construction methods necessary to complete the design for the selected structures.

Figure 9.3: Drop Structure Types







9.2.2.2 Crest and Upstream Hydraulics

Usually, the starting point of drop analysis and design is the designation of the crest cross section at the top of the drop. The upstream slope should be set such that the flows are subcritical and meet the maximum velocity requirements of the applicable channel type as previously discussed in this manual. As flow passes through critical depth near the crest, the crest becomes a control point and upstream hydraulics should be separated from downstream. The critical flow depth at the crest must be calculated and compared with the downstream tailwater depth to verify that the tailwater does not submerge the crest and effectively control the hydraulics at the crest.

With control at the drop crest, upstream water surface profile computations as described in Chapter 4 herein are used to estimate the distance that protection should be maintained upstream, that is, the distance to where localized velocities due to the flow curve are reduced to acceptable values. These backwater computations also yield the maximum upstream flow depth used to set wall abutment and bank heights. For a given discharge, there is a balance between the crest cross section, upstream and downstream flow velocities, and the location of the jump.

9.2.2.3 Water Surface Profile Analysis

Since the purpose of a drop structure is to force lower velocity subcritical flows, both the upstream and downstream channels should be flowing subcritical. Therefore, backwater computations should be completed (as described in Chapter 4 for the upstream and downstream channel reaches. The upstream channel reach will begin at critical depth at the crest of the drop, and calculations will progress upstream. The calculations for the downstream channel will begin at the downstream control point, and progress upstream to the drop structure being designed. This downstream water surface then serves as the ‘tailwater’ condition for the drop structure.

The next step is to determine the location of the hydraulic jump so that the hydraulic jump zone can be sized to adequately contain the zone of turbulence. The determination of the hydraulic jump’s location is usually accomplished using hydraulic analysis software, such as WSPGW or HEC-RAS, or can be determined as described in Chapter 4 . Accommodations for the full length of the hydraulic jump must be provided, plus a reasonable buffer.

Analysis should be conducted for a range of flows, since flow characteristics at the drop can vary with discharge. For example, the 10-year flow may cascade down the face of a sloping drop and form a jump downstream of the toe, whereas the 100-year flow may change the location of the jump, or even totally submerge the drop.

9.2.3 Selection of Drop Structures

There are four major considerations for the selection of the type of drop structure for a particular application: 1) surface flow hydraulic performance, 2) foundation and seepage control, 3) economic considerations, and 4) construction considerations. Other factors that can affect selection are land uses, aesthetics, safety, maintenance, and anticipated downstream channel degradation. Baffle chute, grouted riprap, or loose riprap are the most common to be encountered in Riverside County.

9.2.3.1 Selection Considerations

In addition to hydraulic performance (discussed in Section 9.2.2), a number of other considerations affect the selection of an appropriate drop structure for a particular application.

- Soil and Foundation Condition – Geotechnical investigations should be completed to identify the characteristics of the onsite soils. Silty and sandy soils require detailed analyses for seepage control. Expansive soils require special design techniques to minimize differential movement. Structural design for foundation, walls, and slabs must consider soil bearing capacity, lateral soil and hydrostatic pressures, seepage, and potential scour. A geotechnical/soils engineer must be consulted to review the onsite soils information and geology at the location of each proposed drop structure to ensure that:
    - The proposed structure can be adequately supported by native soils, or to make recommendations for any remediation that may be necessary for a particular structure.
    - Any geotechnical loadings on the drop structure can be determined, including as applicable: soil loads, groundwater loads and earthquake loads.
- Construction Concerns – The selection of a drop structure and its foundation may also be tempered by construction difficulty, access, material availability, etc. Quality control through conscientious inspection is an important consideration. All drop structures must be inspected on a regular basis during construction regarding construction quality and integrity.
- Maintenance Concerns – Drop structures must be monitored on a periodic basis after construction. Issues to be considered in the design include: ease of access to the crest area, continued access across the structure (from upstream to downstream, and vice versa), and maintenance access to the toe of the structure. Potential for sediment accumulation and means for removal must be considered. For sheetpile drop structures, any tiebacks will require testing and potential adjustments, which can require specialized contractors and equipment.
- Sociological Considerations – These include public acceptability issues such as public access and safety (Section 9.9), visual appearance (Section 9.10), etc.

Additional bank and bottom protection may be needed if secondary erosional tendencies are revealed. Thus, it is advisable to establish construction contracts and budgets with this in mind. Use of standardized design methods for the types of drops described herein can reduce the need for secondary design refinements.

9.2.3.2 Surface Flow Hydraulic System

The primary consideration for the selection of a drop structure should be functional hydraulic performance. The surface flow hydraulic system combines channel approach and crest hydraulics, sloping drop hydraulics, and downstream tailwater conditions. Hydraulic analysis procedures are presented in Section 9.3.2. Additional guidelines are also contained in Section 9.2.4.

9.2.3.3 Foundation and Seepage Control Systems

The hydraulic engineer must calculate hydraulic loadings which can occur for a variety of conditions such as interim construction conditions, low flow, and flood flow. The soils/foundation engineer couples this information with the onsite soils information. Both work with a structural engineer to establish final loading diagrams, and selection and sizing of structural components. This Section presents information relevant to hydraulics but refer to geotechnical and structural books for related information.

The District requires all upstream cutoff walls to be constructed of concrete.

9.2.3.4 Construction Considerations

The selection of a drop and its foundation may also be tempered by construction difficulty, location, access, and material availability/delivery. Table 9.2 lists construction considerations for key drop structure materials. Additional discussion of construction concerns is included with the design guidelines for each drop type in the following Section.

Table 9.2: Quality Control Measures and Concerns of Drop Structure Components

Type	Quality Concerns	Quality Control Measures and Inspection
Concrete	The major concern is strength and ability to resist weathering and abrasion. Aggregate strength and durability are important.	Preconstruction items include review of shop drawings for formwork patterns and ties, concrete design mix and related tests, color additives or coatings and architectural treatments such as form liners, handrails, and fences. During construction there are numerous items that require checking, including: formwork, tie placement, weep holes and drains, form release coatings and form cleaning before concrete placement, form removal, concrete placement and testing, weather protection, sealants, tie hole treatment, concrete finish work, and earthwork, especially that related to seepage control.
Reinforcing Steel	Usually not a problem unless the wrong grade of steel is brought to job, or site conditions are conducive to corrosion problems. Epoxy coated reinforcement can be specified for critical conditions.	Preconstruction items include review of shop drawings for reinforcing steel. During construction rebar placement needs to be checked. Sufficient steel cover when high velocities occur, see Section 6.2.8.
Riprap and Rock	See Section 8.7.5.3	See Section 8.7.5.3

Type	Quality Concerns	Quality Control Measures and Inspection
Grout	Cement content and type, aggregate, and water content are important considerations for strength and durability.	The key to success with grouting is to have sufficient voids to allow for full grout penetration and to pump and place the grout using a grout pumper with a nozzle that can penetrate to the subgrade, to vibrate using a "pencil vibrator" to assure complete filling of the voids, to have good control of the grout mix (too wet creates shrinkage cracks and stability problems on slope, too dry leads to poor penetration), and to place the grout to the desired thickness. A minimum grout thickness is needed to counteract uplift forces. However, placing too much can submerge the rock, eliminating the energy dissipation that exposed rock provides. During grouting, it is important to protect the weep drains. With care, one can avoid getting grout on the top of the rock. Any spillage should be washed off immediately. A wood float leaves a smooth finish, and the "pencil vibrator", which is preferred, will generally leave a satisfactory appearance with some touch-up. Full time inspection during grouting and the rock placement depending upon the performance of the contractor and the aesthetic appearance desired.
Sheetpile Drop Structure	Sheetpile comes in many configurations and, in particular, joint details. It requires geotechnical, structural, and hydraulic expertise, as well as pile driving experience during construction. Sheetpile Drop Structures are generally not allowed on facilities to be maintained by the District.	Inspection is required to ensure that piling is driven to the design depth, or keyed into bedrock, if required. Underground obstructions can create problems with driving. If piling becomes separated at the joints during installation or doesn't achieve the required depth, excessive surface flow, and/or structure failure can result. Sheetpile Drop Structures are generally not allowed on facilities to be maintained by the District.
Subdrains	Permeability and gradation of media, reverse filter characteristics and compatibility with in-situ materials, pipe, and other hydraulic components.	Gradation analysis of in-situ materials and proposed filter media are advisable. Fabric materials should be used with caution to insure that plugging will not occur. Piping should comply with specification and be double checked for suitability for the particular application. A subdrain may be required depending on the soil and water table condition and as directed by the engineer.

9.2.4 Design Guideline – Sloping Rock Drop

Sloping rock drop structures have gained popularity due to their design aesthetic and successful application. The sloping rock drop is designed to operate as a hydraulic jump dissipater, although some energy loss is incurred due to the roughness of the rock slope. The quality of rock used and proper grouting procedure (if grouted rock is used) are very important to the structural integrity. The main design objectives are to maintain structural integrity and to contain erosive turbulence within the downstream riprap blanket.

9.2.4.1 Grouted Rock

Grouted rock drops must be constructed of uniform size boulders grouted in place through the approach, and sloping face. However, because the structure is comprised of a structural slab with two components (boulders and grout), great care must be taken to design the structure to withstand uplift and to specify boulder and grout material to assure full quality control in the field. Seepage analysis is required to determine a compatible combination of cutoff depth, location of the toe drain and/or other drains, and the thickness of rock and grout. Problems with rock specific gravity, durability and hardness are of concern. Gradation problems are largely eliminated because the boulders are specified to meet minimum physical dimensions and/or weights, which is much easier to observe and enforce in the field than with graded riprap.

The handling of the large rock boulders requires skilled work force and specialized equipment. Equipment like logging tongs, and specially modified buckets with hydraulically powered "thumbs" have been used in recent years and have greatly improved quality and placement rates. The careful placement of stacked boulders, so that the upstream rock is keyed in behind the downstream rock, and placed with a large flat surface horizontally, has been shown to be successful. Prior to placement of grout, the rocks should be pressure washed, including down into the voids to ensure that dirt is removed so the concrete can adhere to each stone.

The greatest danger lies with a "sugar coated" grout job, where the grout does not penetrate the voids between the rock and the subgrade, leaving a direct piping route for water under the grout. This can easily occur when attempting to grout graded riprap, thus the need to use individual boulders that are larger in diameter than the grout layer so that the contractor and the inspector can see and have grout placed directly to the subgrade. The best balance appears to be boulders 33 to 50 percent greater in size than the grout thickness, but of an overall weight sufficient to offset uplift. Also, when holding grout to this level, the appearance will be much better.

The grout should have a minimum 4,000 psi compressive strength at 28 days, stone aggregate with a maximum dimension of one-half inch and a slump within a range of 4 to 7 inches. The water/cement ratio should not exceed 0.48.

Grouted rock drop structure shall include appropriate structural analysis and analysis of geotechnical factors such as seepage. Weep drains should be considered for seepage and uplift control. Weep systems require special attention during construction. The boulders can crush the pipes and alignment of the pipes between the boulders can be difficult.

9.2.4.2 Loose Rock

The loose angular riprap d_{50} equations for the sloped drop structure at different slope ranges have been developed by Robinson, et al. (1998). As indicated by Robinson, et al. (1998), an appropriate safety factor should be applied when using these equations. With a safety factor of 1.5, the loose angular riprap median size equations are simplified from Robinson, et al. (1998). The specific weight for rock drop sizing equation development is 162 lb/ft^3. It is a weighted-average value of the riprap D_{50} for the 38 experiments published in Robinson, et al. (1998). So, the specific weight of riprap should be at least 162 lb/ft^3 in the following equations.

$$d_{50} = 2.12q^{0.529}S_o^{0.794} \qquad 0.02<S_o<0.10 \qquad (9.1)$$

$$d_{50} = 0.69q^{0.529}S_o^{0.307} \qquad 0.1\leq S_o\leq 0.4 \qquad (9.2)$$

where:

d_{50} = the median diameter (ft)
q = unit discharge (cfs/ft), (discharge divided by the width where the width is defined as the wetted area divided by the flow depth. For the purposes of this equation, the actual flow depth calculated on the face of the slope of the drop structure should be used. For approximation, the flow depth can be estimated from the Manning's equation based normal depth or the maximum flow depth from HEC-RAS).
S_0 = longitudinal slope of the face of the drop structure (ft/ft)

It should be noted that these two equations are for loose riprap on the slope of the structure. For downstream of the sloped drop structure, Equation (9.3) should be used. The thickness for the riprap layer on the slope should be at least $2d_{50}$. A granular filter should be used beneath the riprap layer. The design for rock chutes and downstream energy dissipators can be found in Lorenz, et al. (2000).

9.2.4.3 Approach Apron

The boulder drop structure typically has a 10-ft trapezoidal riprap approach section immediately upstream of its crest. The approach apron is provided to protect against the increasing velocities and turbulence that result as the water approaches the sloping portion of the drop structure. The width of the approach apron and the side slopes should match the upstream channel, and the height of the boulder channel sides shall be equal to the depth of water in the upstream channel plus the required freeboard as described in Section 8.3.7.

A concrete cutoff wall shall be placed at the upstream side of the approach apron to reduce or eliminate seepage and piping through the structure. The depth of the cutoff wall shall be at least as deep as the finished grade at the toe of the drop or 1 foot deeper than the full depth of the riprap layer, whichever is deeper.

9.2.4.4 Drop

The slope of the drop structure shall not be steeper than 4H:1V. Slopes flatter than 4H:1V usually increase expense, but some improvement in appearance may be gained. The side slopes and bottom width of the drop should be as close to the upstream as possible. The grouted/ungrouted boulders shall extend up the side slopes a height of the tailwater depth plus freeboard as projected from the downstream channel or the critical depth plus 1 foot, whichever is greater.

9.2.4.5 Exit Apron

The exit apron is necessary to minimize any erosion that may occur due to secondary currents and should extend a minimum of 30 feet downstream of the drop toe or to where velocities would exit the apron in a non-erosive fashion (i.e., 6 ft/s or less for typical soils), whichever distance is greater. The bottom width and side slopes of the exit apron shall be the same as the downstream channel. The apron sides shall extend to a height equal to the tailwater depth plus the required freeboard.

9.2.4.6 Downstream of Drop Structure

The loose riprap d_{50} for channel bed protection downstream of a drop structure is:

$$d_{50} = 0.0372 V_a^2 \left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \tag{9.3}$$

where:

d_{50} = the median diameter (ft)
V_a = average velocity (ft/s)
γ_S = specific weight of stone (lb/ft^3)
γ_W = specific weight of water (lb/ft^3)

This equation is also a simplified Isbash equation with C=0.86 and 0.0 degrees of bank angle.

9.2.5 Design Guideline - Baffle Chute Drops

The USBR Type IX Baffled Apron is allowed by the District and the design procedures are included in Chapter 7 of HEC-14.

The District typically does not permit depressed stilling basins due to vector issues associated with standing water.

The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of the drop, which is commonly referred to as baffled apron or baffle chute drops. Design of this type of structure is beyond the scope of this manual. Information about baffled apron/ baffle chute drops proper application and design can be obtained from several sources, including: *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka, 1984), HEC-14 *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, 2006), and *Design of Small Canal Structures* (USBR, 1974). Another reference is *Baffled Apron as Spillway Energy Dissipator* (Rhone, 1977), which evaluates higher design discharges, and entrance modifications to reduce the backwater effect caused by the baffles.

9.2.5.1 Sloping Concrete Drops

The hydraulic concept of these structures is to dissipate energy by formation of a conventional hydraulic jump, usually associated with a reverse current surface flow as the supercritical flow down the face converts to subcritical flow downstream.

General Hydraulic Design Procedure:

Analysis of channel approach and crest hydraulics generally follows the guidelines presented in Section 9.2.2. Once water surface profiles have been determined, including tailwater determination and supercritical water surfaces down the sloping face, seepage uplift forces must be evaluated. Net uplift forces vary as a function of location along the drop, cutoff measures, drain gallery locations, and water surface profiles through the basin.

For a stable structure, net uplift force from seepage must be countered by net forces in the downward direction. For a smooth concrete chute, downward forces are the buoyant weight of the concrete structure and the weight of water (a function of the depth of flow). Significant pressure differentials can occur with a combination of high seepage forces and shallow supercritical flow.

Construction Considerations:

There may be applications where sloping concrete drops are advantageous, but generally other drops such as baffle chutes are more appropriate for a wider range of applications. The design guidance provided by the literature is clear and relatively easy to use, but the implementation is often difficult or impractical. This basically has to do with providing basin depth without creating a maintenance problem and less flexibility in adapting to varying bed conditions.

The integrity of the cutoff is important as seepage and resultant uplift forces are key concerns. Uncontrolled underflow could easily lift a major concrete slab.

The stilling basin should be designed to drain completely to eliminate nuisances related to ponded water, such as mosquito breeding and sediment/debris accumulation.

Considerations relating to general concrete construction are the same as discussed. Public acceptability is likely to be low in urban areas, as the sloping concrete face is inviting for bicyclists, roller skaters, and skateboard enthusiasts.

9.2.6 Other Types of Drop Structures

There are numerous other types of drop structures for specific applications in drainage design. Riprap drops and baffled aprons are the most common to be encountered in Riverside County. Drop structures other than riprap or baffled aprons may be acceptable on a case-by-case basis with approval of the governing agency.

9.2.7 Grade Control Structures

Grade control structures are similar to drop structures. However, instead of stabilizing natural channels and other unlined channels to *prevent* future erosion or headcutting, these structures are designed to *prevent* future erosion or headcutting. Here, headcutting is defined as the scouring of the channel bed proceeding from a downstream to upstream direction.

Grade Control Structures are not covered in detail in this manual, however the basic design procedure for grade control structures is to: 1) determine a stable slope based on hydrology, hydraulics, sediment load and characteristics, and 2) determine spacing of the grade control structures based on the difference in slope between the natural and projected stable slope, and 3) design that will stabilize that future grade. Sediment transport and estimating scour depth is beyond the scope of this manual, refer to Los Angeles County Department of Public Works sedimentation manual for guidance. The longevity of the structure is dependent upon the depth of toe down (among other things), which must exceed the depth of scour that will exist when that future eroded state exists. The potential for seepage cutoff must be assessed for hydrostatic pressure and the potential failure of the structure foundation due to "piping" of the underlying soils. If an issue, the appropriate engineered solutions should be employed in the design. These solutions include the use of geotextile filter fabrics to prevent soil loss and small diameter PVC pipes to relieve hydrostatic pressure. In any case, appropriate access to grade control structures is necessary to permit intermittent maintenance.

9.3 ENERGY DISSIPATION STRUCTURES FOR CONDUITS

9.3.1 General

This Section is applicable to both culvert and storm drains. Energy dissipation structures are used to reduce the velocity of flows to acceptable levels. There are two primary reasons this may be necessary:

- Velocities within the conduit are high enough to damage the conduit itself.
- Velocities being discharged from the culvert may be damaging to downstream systems.

Within the conduit, velocities must be verified to not exceed the maximum permissible velocity of the conduit material, as described in Section 6.2.8. If the velocities are too high, and changes in profile or conduit size/type cannot resolve the issue, internal energy dissipators may be able to help reduce the velocity.

At the location where a storm drain or culvert discharges to a natural watercourse, unlined channel or other erodible surface, the water must be slowed down and spread out to match the existing (pre-project) conditions. External energy dissipators can accomplish this goal.

> Important Note – The need for energy dissipation at a storm drain or culvert outlet functions in tandem with the topics discussed in Chapter 10 . Where a development project has the potential to increase the overall quantity of runoff (i.e., increases in peak flow, etc.), Chapter 10 provides required methods to reduce the *quantity* of runoff to acceptable levels. Having accomplished that goal, it is still necessary to install energy dissipation at the outlet as described in this Section to reduce the *velocity* of the discharge. However, installation of these energy dissipators is never a substitute for compliance with Chapter 10 .

There are many methods that have been developed for energy dissipation, which range from complicated to simple, and each has different applicability, cost, and maintainability. This Section provides guidance for the types that are most commonly used in Riverside County due to their simplicity and ability to function in our semi-arid and arid environment.

The designer is encouraged to investigate referenced sources for further discussion of energy dissipators. If the project requires an energy dissipater that is not listed by this manual, the designer shall contact the local governing agency prior to proceeding with design. This will enable the local governing agency and designer to coordinate design, submittal, inspection, and maintenance requirements.

9.3.2 General Design Process

Where velocities need to be reduced within a conduit, there are two strategies that can be used:

- Reduce the slope of the conduit, if possible. Changes to the conduit size or type can also help, particularly in combination with reductions to the slope of the conduit.

- Install 'Internal Dissipator' strategies within the conduit to increase the effective roughness. This will also have impacts on the size of the conduit but can help lower velocities where changes to the slope may not be possible. Strategies for internal dissipation are described in Section 9.3.3.

At a conduit outlet, there are two goals that must be met:

- The velocity must be reduced to levels that will either not be erosive, or not exceed the natural velocity of flows downstream of the outlet.
    - To determine the acceptable non-erosive velocities, refer to Table 8.2.
    - To determine the natural velocity of flows downstream of the outlet, a hydraulic analysis of the existing area must be performed using the methods described in Chapter 4 .
- The water must be spread out to mimic the existing flow conditions.

These two goals at the outlet can be met by designing an appropriate energy dissipator strategy. Sometimes a combination of strategies can be more effective. For example, using internal dissipators near the outlet may reduce the size of external dissipators at the outlet. Sections 9.3.3 through 9.3.5 describe some strategies that have been used in Riverside County. If alternative strategies are desired or necessary for a particular project, discuss the intended strategy early on with the governing and maintaining agency.

9.3.3 Internal (Integrated) Dissipators

Strategies for Internal Dissipators are described in FHWA HEC-14, Chapter 7. Table 9.3 below summarizes application and constraints on their use for District-maintained facilities. The District's preferred approach is to use velocity rings (a form of increase resistance) as described in HEC-14, Section 7.2.

Table 9.3: Application of Internal Dissipators

Type	Application and Constraints
Tumbling Flow (HEC-14, Section 7.1)	• These measures cannot be used in conduit reaches that are anticipated to receive sediment or debris loads accumulations, as the constructed elements inhibit the ability for required maintenance. • The size of the roughness elements must not exceed the following: ○ For pipes: the relationship between the inner diameter of the ring, Di, to the pipe diameter, D, is 85-90% meaning the relationship of roughness height, h, to pipe diameter, D, is 10-15%.
Increased Resistance (HEC-14, Section 7.2)	• These measures cannot be used in conduit reaches that are anticipated to receive sediment or debris loads accumulations, as the constructed elements inhibit the ability for required maintenance.
Broken-Back Culverts	• This approach can be helpful to minimizing outlet velocities but increases the slope (and likely the velocity) of flows in the steeper

Type	Application and Constraints
(HEC-14, Section 7.4.1)	reach. These higher velocities must be compared to allowable velocities for the conduit type. • The guidance in Section 7.2.2.12 of this manual must be followed.
Outlet Drops and Weirs (HEC-14, Section 7.4.2 and 7.4.3)	• This system as described in HEC-14 has limited applicability because water overflowing the weir will return to supercritical flow, which will likely still require energy dissipation downstream of the outlet. • Use of this system on a district-maintained system will require pre-approval and case-by-case considerations such as but not limited to: ○ These measures cannot be used in conduit reaches that are anticipated to receive sediment or debris loads accumulations, as the accumulations will inhibit the function of the weir and limit accessibility for maintenance. ○ Maintenance Access Vaults must be installed upstream of the weir to allow access for maintenance access. ○ The weir must be designed with a full-height slot to drain ponded water.

The District prefers the use of Velocity Control Rings as described in Section 7.2 of FHWA HEC-14. These rings increase resistance through the length of pipe, thus reducing velocities along the length of the storm drain where they are installed. The use of velocity rings in reinforced concrete boxes is prohibited by the District. The culvert velocity reduction by internal energy dissipators (velocity control rings or roughness elements) force the hydraulic jump to occur within the culvert, therefore, reducing the size and extent of costly outlet structures.

Velocity control rings shall be modified to facilitate drainage behind the rings and prevent ponding within the culvert. The use of velocity control rings is intended for applications where velocities within the pipe are 20 fps and greater and where no significant bedloads are anticipated, or where other methods of energy dissipation are impracticable. Standard Plans for Public Works (Greenbook) Standard Plan 383-2 (Public Works Standards, Inc., 2013) has an example of a precast velocity control ring, which includes a drainage notch to prevent ponding within the culvert.

9.3.4 Riprap Basins and Aprons

Riprap Basins and Aprons are installed at the outlet of a culvert or storm drain to reduce the velocities and spread-out flows. The design of these two strategies are described in FHWA HEC-14, Chapter 10. Table 9.4 below summarizes application and constraints on their use for District-maintained facilities.

Table 9.4: Application of Riprap Basins and Aprons

Type	Application and Constraints
RipRap Basins (HEC-14 Section 10.1)	• These structures use Riprap in a specifically designed basin configuration and includes a method for direct calculation of the velocity exiting the RipRap Basin, which can then be compared to the allowable velocity for the receiving stream.

Type	Application and Constraints
	• The basin must be sized and constructed following the guidance provided in HEC-14.
RipRap Aprons (HEC-14, Section 10.2)	• These structures are simpler to build than a RipRap Basin, but do not include a method to calculate the velocity exiting the apron, for comparison to the design criteria. The velocity exiting the apron must be calculated via other methods, such as described in Chapter 4 of this manual, and then compared to the allowable velocity for the receiving stream. • The longitudinal slope of the apron should be zero percent to effectively reduce the velocity. • HEC-14 provides a method for determining the D50 of the Rock. The results of those calculations should be compared to the methods described in Section 8.7 and the larger size used. The smallest D50 the District will maintain is ¼ Ton (Class V). If culverts outfall to a channel as a side drainage system, the riprap must also be large enough to withstand the flow in the main channel and be inset into the ground so as to not adversely impact flow within the main channel. HEC-14 and RCTD Standard No. 314 provide relationships for the length and width of the apron. The length and width should be no shorter than those requirements and must be extended as needed to meet the design criteria for velocity reduction and spread of flows.
District Standard Drawing JS233	• Where a storm drain discharges to a soft-bottom engineered channel, District Standard Drawing JS233 shall be used.

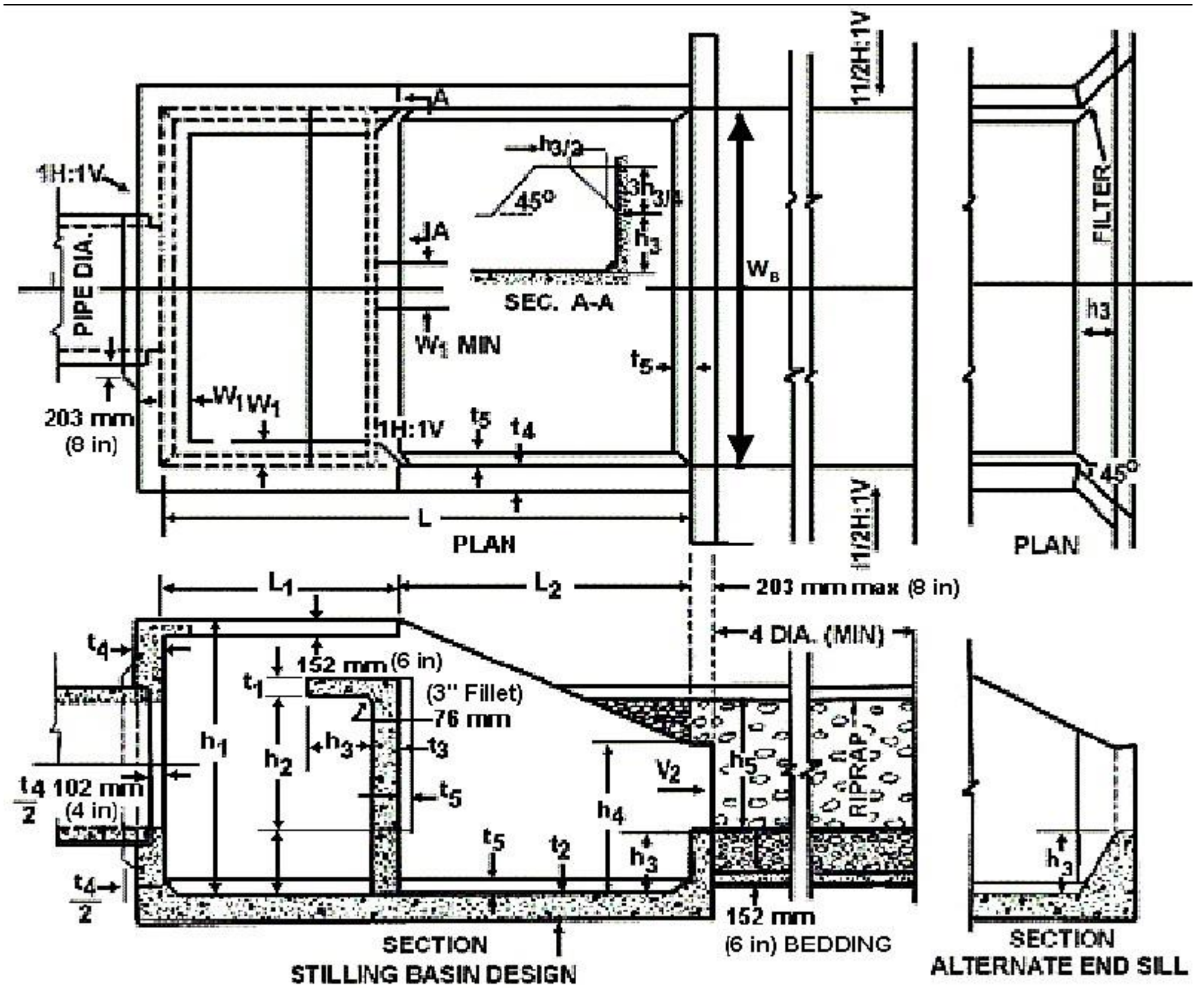
9.3.5 Impact Basin

Scour protection alone cannot be used to reduce discharge velocities to be non-erosive when exit velocities are greater than 20 fps or where the allowable footprint for outlet energy dissipation is smaller than what would be required with riprap alone. In this case, an impact basin style energy dissipater may be utilized instead.

Design standards for the impact basin (also known as impact structure or stilling basin) depicted on Standard Plans for Public Works (Greenbook) Standard Plan No. 384-3 (Public Works Standards, Inc., 2013) are based on the USBR Type VI Basin shown in Figure 9.4. The original USBR basin was modified on SPPWC Standard Plan 384-3 to allow drainage of the basin during dry periods, which reduces the likelihood of vector issues thereby enhancing the usefulness of the basin in urban environments. Section 9.4 of FHWA HEC-14 provides a complete discussion of the use and design of USBR Type VI impact basins. The width of the structure is based on discharge from the storm drain or culvert; this width must be specified on drawings. Drivable access to the outlet shall be provided for maintenance. Fencing shall be provided as required by Cal OSHA.

Riprap should be placed downstream of the dissipater with a length equal to at least four conduit widths. Toe down the downstream end of this rock, with consideration to specific application, including outlet velocity, streambed material, and downstream slope.

Figure 9.4: USBR Type VI Impact Basin
(USDOT, 2006)



9.4 *WEIRS AND ORIFICES*

9.4.1 About Weir Flow

Weirs are structures that are constructed to control the water level in a watercourse or reservoir and regulate discharge to the downstream system. Common applications for weir flow in stormwater drainage are:

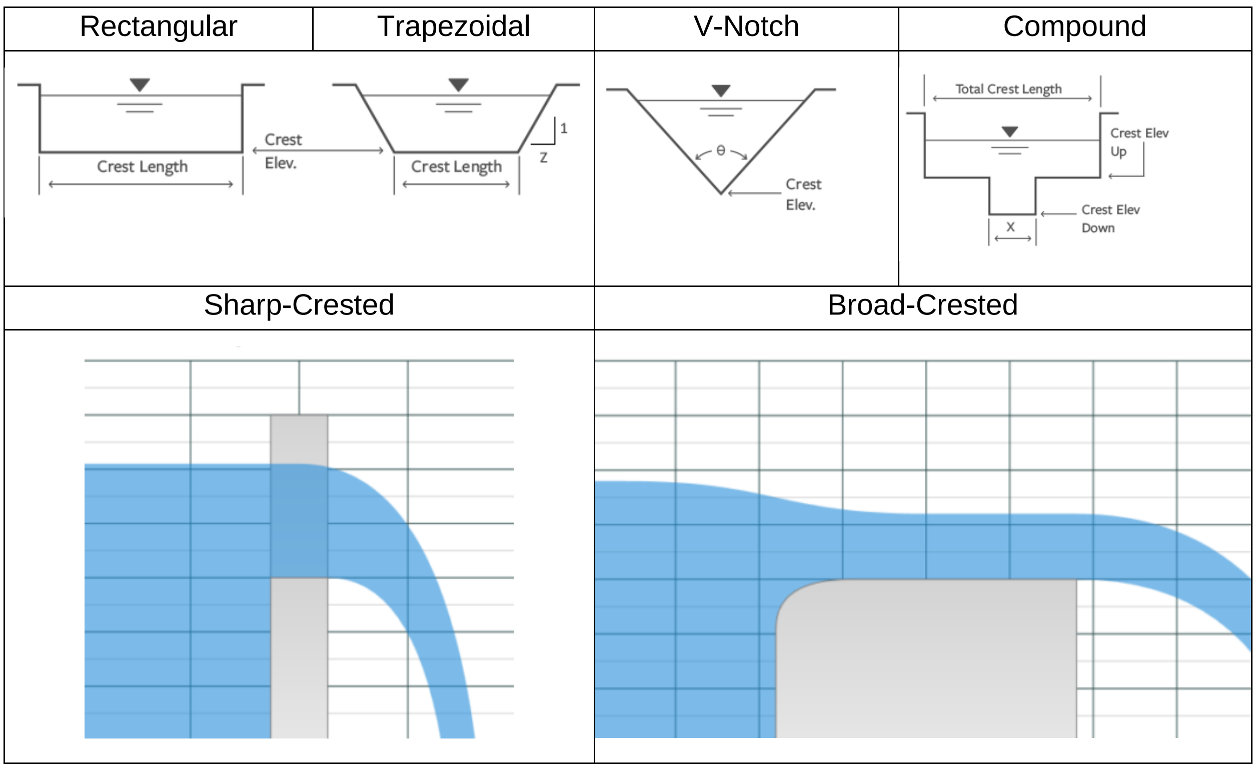
- Overtopping of a basin or dam such as via a spillway
- Overtopping of a roadway, such as when the capacity of a culvert is exceeded
- Scenarios where water falls over a clear ‘threshold’ even if the structure itself is not a conventional weir. Examples can include flow into a catch basin or drop inlet, or ‘inlet control’ flows exiting a basin into a culvert pipe or box.

This Section provides methods for the hydraulic calculations for the amount of water and depth of water flowing over certain types of weirs. This manual does not cover the design of the structure of a weir nor the chute downstream of a spillway. When upstream flows are significantly deeper than the height of the catch basin, pipe or box opening, weir flow equations do not apply, and the outlet will function as an orifice.

9.4.2 <u>Hydraulic Analysis of Weirs</u>

Weirs covered in this manual can be generally classified as sharp crested or broad crested and have sections including rectangular, trapezoidal, V-notch, and compound. The primary difference between sharp-crested and broad-crested weirs is the thickness of the weir (in profile) relative to the depth of water passing the crest. Where the crest thickness is greater than 6/10 the depth of flow over the weir, the weir can be considered broad crested (Simon, 1981). Sharp-crested weirs have a relatively thin crest such that water will tend to develop a nappe as it flows over the crest. B-notch weirs are a particular type of sharp-crested weir with a triangular cross-section.

Figure 9.5: Weir Types



9.4.2.1 Weir Formulas

The formulas in Table 9.5 shall be used for the calculations of the various weir types.

Table 9.5: Weir Equations

Weir Type	Equation		Eqn. No.
Rectangular	Suppressed	Contracted	(9.4)
	$Q_w = C_w L H^{3/2}$	$Q_w = C_d L' \, {}^{2}/_{3} \sqrt{2g} H^{3/2}$	
Trapezoidal	The flow over each side slope of the trapezoid is: $Q = \left({}^{2}/_{5}\right) \times C_w \times Z \times H^{5/2}$		(9.5)
	The total flow rate over a trapezoidal weir is therefore: $Q_w = \left[C_w L H^{3/2}\right] + 2 \cdot \left[\left({}^{2}/_{5}\right) \times C_w \times Z \times H^{5/2}\right]$		(9.6)
V-Notch	$Q_w = 2.54 \cdot \tan\left({}^{\theta}/_{2}\right) \cdot H^{5/2}$		(9.7)
Compound	Combine Equations (9.4) through (9.7) as applicable		

where:

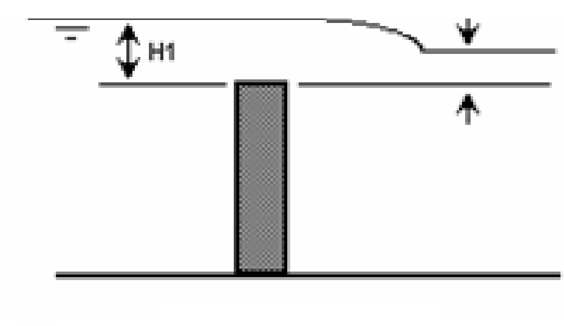
Qw = discharge over the weir (cfs)
Cd = coefficient of discharge
Cw = weir coefficient
L = crest length (ft)*
L' = $[L - 0.1(n)H]$; modified crest length (ft)
H = head over the crest elevation (ft) For broad-crested weirs, the head is measured at least 2.5H upstream of the weir
n = number of contractions (1 or 2)
Z = ratio of horizontal to vertical distance for weir side slopes
θ = angle between the two slopes of the V-notch weir (degrees)

*Where trash racks or other barriers are installed within an area otherwise flowing as a weir, L shall be reduced to a) account for the clear space between the bars/obstructions, and b) account for clogging.

9.4.2.2 Adjustment for Submerged Weirs

Equations (9.4) through (9.7) apply where the tailwater (water surface downstream of the spillway is below the crest elevation of the weir. It is important to note that downstream water surface elevation (tailwater) must be analyzed by appropriate methods (see Chapter 8) to assess potential for submergence of any weir.

Figure 9.6: Submered Weir



If the tailwater rises above the crest of the weir, the submerged condition will reduce the flow rate over the weir. The equation for the submerged sharp-crested weir flow is:

$$Q_s = Q_r\left(1 - \left({}^{H2}/_{H1}\right)^{1.5}\right)^{0.385} \qquad (9.8)$$

where:

Q_s = Submerged flow (cfs)
Q_r = unsubmerged flow from standard weir equations (cfs)
$H1$ = upstream head above the crest (ft)
$H2$ = downstream head above the crest (ft)

9.4.2.3 Weir Coefficient

The selection of the weir coefficient, C, is a function of numerous factors including the total head on the weir, the vertical height of the weir, inclined faces of the weir (both upstream and/or downstream), submergence conditions, and for broad-crested weirs – the breadth of the weir. Care must be taken in selecting the value of C and in applying appropriate correction factors to C depending upon the structure configuration and flow conditions.

Table 9.6: Weir Coefficient

Weir Type	Weir Coefficient, C
Broad-Crested	Broad-crested weirs have widely varying physical conditions which significantly affects the value of the weir coefficient. The normal range of C is from about 2.4 to about 3.5, however, use of values in excess of 3.1 must be carefully analyzed and are generally not recommended. A discharge coefficient of 3.0 is typical for flow over roadway embankments without backwater (Bureau of Public Roads,1978). Performing a sensitivity analysis for the expected range of weir coefficients (such as using a value 0.1 over and 0.1 under the calculated value) is suggested to confirm the design. See Section 7.4 for adjustment to C for roadway embankments subjected to submergence. The head, H, is measured at least 2.5H upstream of the weir for broadcrested weirs.
Sharp-Crested	For sharp-crested weirs, the weir coefficient can range from about 3.2 to an excess of 5.0. The Rehbock equation (Equation (9.9)) (Chow, 1959, page 362) is often used to estimate C for sharp-crested weirs: $$C = 3.27 + 0.40H/h_w \qquad (9.9)$$ where H is the measured head and h_w is the height of the weir. That equation is valid for H/h_w up to 5 but can be extended to $H/h_w = 10$ with fair approximation. Values of C in excess of 5.0 should not be used without careful deliberation of all factors including consequence of overestimated capacity. Typical C values are in the lower end of the aforementioned range. It is important to note that this discussion assumes that the nappe of water over the sharp-crested weir is fully aerated. Insufficient aeration will result in undesirable performance, including pressure differential on the structure, unsteady and pulsing discharge over the weir, and increase in spillway discharge. Braeter and King (1976) provides useful tables in selecting appropriate values for C.

Ogee shaped spillways can offer the best hydraulic performance, however, the cost of such spillways is usually greater than other comparable weir types. Ogee spillways must be designed and analyzed by appropriate methods, such as those enumerated in the *Design of Small Dams* (USBR, 1987, p353 and 366-367).

9.4.3 About Orifice Flow

Orifices are holes or openings through the side or bottom of an impoundment area that are entirely submerged on the upstream side, and entirely free-flowing on the downstream side. In stormwater drainage, common applications for orifice conditions include basin outlet conduits, particularly when the water in the basin is quite deep. Orifice conditions may also exist for catch basins or other inlet structures where the surrounding area is a sufficient sump that allows water to pond to a level that exceeds the height of the opening.

It should be re-emphasized that for a discharge to function as an orifice, the opening must be entirely submerged on the upstream side, and entirely free-flowing on the downstream side.

- If the downstream side of the orifice is submerged or affected by a backwater, flow through the opening is limited by the capacity of the downstream system and will not function as an orifice. In this case, it is more appropriate to analyze the opening as an abrupt transition loss, using the methods described in Chapter 6 .
- On the upstream side, if the opening is only partially submerged, flow exiting through the opening will not function as an orifice and should instead be analyzed as a contracted rectangular weir using the methods described above.

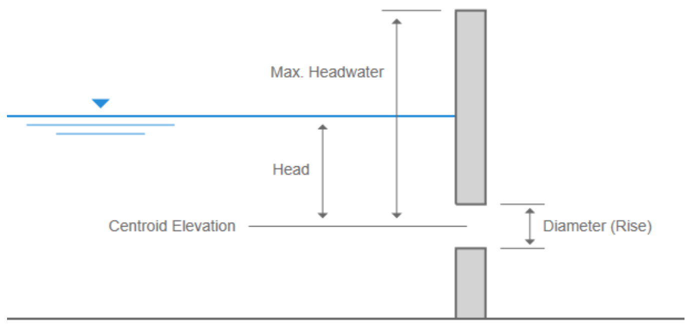
Many detention basin outlets structures incorporate a vertical orifice opening in either an orifice plate or a riser. For typical detention basins, the orifices are being used to meter discharge for smaller storm events, rather than the design storm, and the discharge capacity of the orifices are not significant in relation to the overall functionality of the basin. In these cases, the orifices are small (i.e., up to 6" diameter) and the rating curves may omit the partially submerged condition. That is, the rating curve would have stage/storage/discharge data at the orifice invert elevation and the top of orifice elevation with no intermediate values utilized.

For the unsubmerged condition, the effective head d is measured from the centerline of the orifice to the upstream water surface elevation. For the submerged condition, 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. Figure 9.8 summarizes orifice discharge coefficient for different edge conditions.

When pipes are utilized as a basin outlet, those less than or equal to 1 foot in diameter may be analyzed as orifices. Pipes larger than 1 foot in diameter are more appropriately analyzed as culverts (see Chapter 7). Flow through multiple orifices may be computed by summing the flow through the individual orifices.

Figure 9.7: Orifice Diagram



9.4.4 Hydraulic Analysis of Orifices

9.4.4.1 Orifice Equation

The following equation shall be used for the calculation of orifice flow:

$$Q_o = C_d \cdot A_o \cdot \sqrt{2 \cdot g \cdot d} \tag{9.10}$$

where:

Q_o = discharge through the orifice (cfs)
C_d = orifice discharge coefficient
A_o = clear opening area for the orifice (ft^2)
g = gravitational constant = 32.2 (ft/s^2).
d = head as measured from the centroid of the opening (ft)

9.4.4.2 Orifice Discharge Coefficient

The discharge coefficient can vary widely depending on the shape of the edge of the opening. Figure 9.8 below shows different types and the corresponding C_d.

Figure 9.8: Orifice Discharge Coefficient

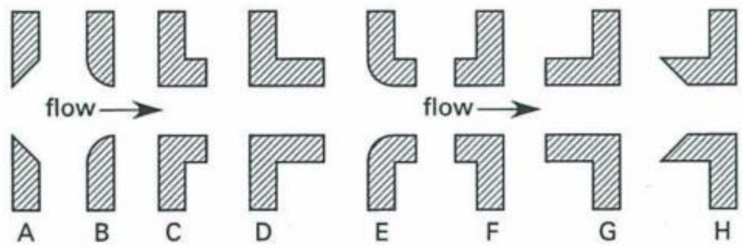


illustration	description	C_d
A	sharp-edged	0.62
B	round-edged	0.98
C	short tube[a] (fluid separates from walls)	0.61
D	sharp tube (no separation)	0.82
E	sharp tube with rounded entrance	0.97
F	reentrant tube, length less than one-half of pipe diameter	0.54
G	reentrant tube, length 2 to 3 pipe diameters	0.72
H	Borda	0.51

9.4.5 Compound Rating Curves

When a combination of multiple weirs, orifices, or both are installed, a compound rating curve is developed to determine the outflow associated with a range of depths that engage each orifice/weir as the depth increases. This is of particular use in storage/detention routing (see also Chapter 10).

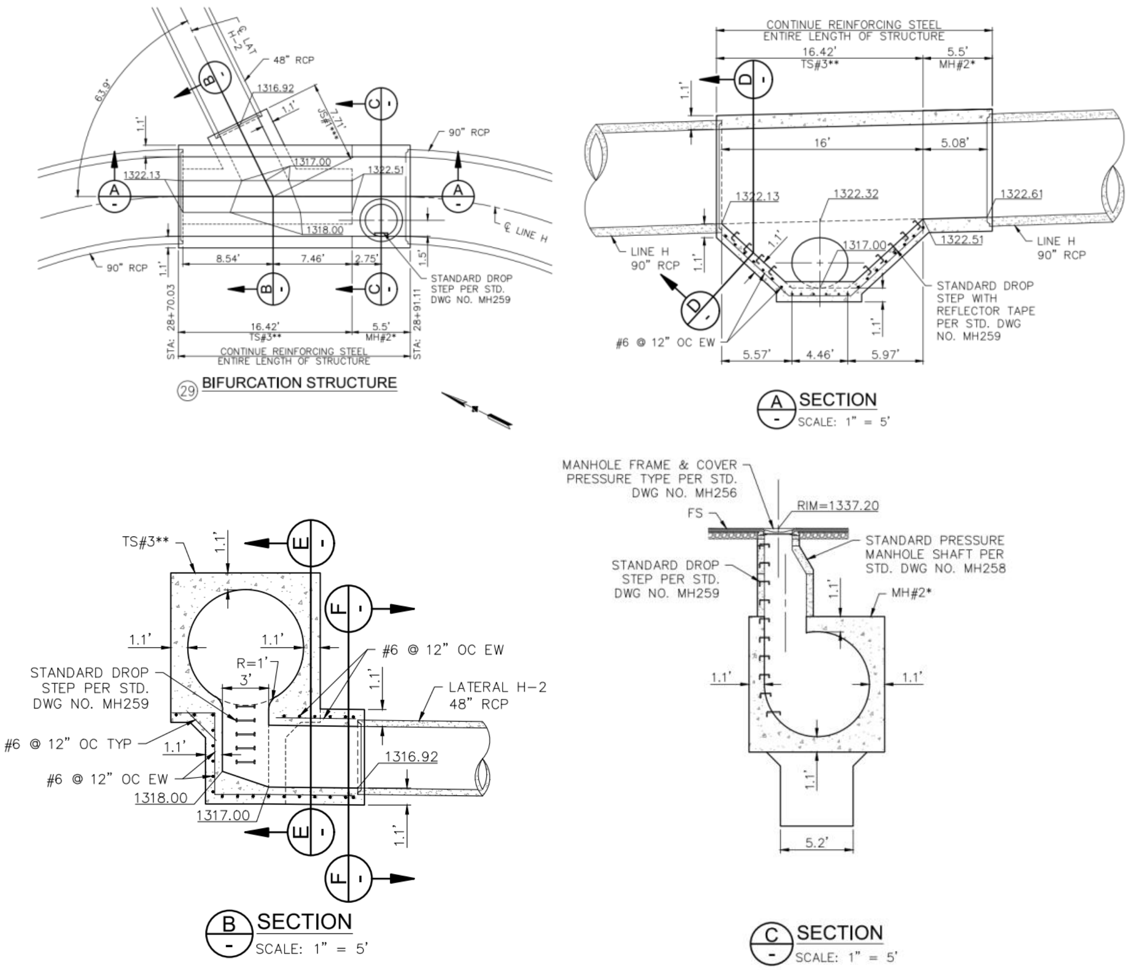
9.5 BIFURCATION STRUCTURES

It may occasionally be necessary to divert part of the flow in a channel. For example, the designer may need to divert a portion of the flow to a stormwater basin, or the downstream right of way may be too narrow to accommodate the full flow and a portion of the flow may have to be diverted to another outfall point. In these instances, the designer will have to provide a "splitter" or bifurcation structure to apportion the flow in the appropriate direction.

There are three principal types of splitters: 1) Low Flow Diversion, 2) Parallel Flow Splitter, and 3) Side Channel Weirs.

Low Flow Diversion – One is a low flow diversion, where the invert of a larger channel or conduit may be constructed with smaller low flow channel that diverts water out of the channel and into an adjacent pipe. This is commonly done to divert lower flows into a basin, typically for water quality or sedimentation purposes for smaller storm events. An example on a District project was constructed on the Lakeland Village Line H Storm Drain, where an offline basin was constructed. Smaller storm flows were diverted from the invert of the channel into this basin to allow deposition of suspended sediments. See examples of this low flow diversion structure below:

Figure 9.9 : Lakeland Village Line H Diversion Structure



The hydraulics for a low flow diversion will depend on the circumstances, but generally the hydraulics for the low flow conduit should follow the guidance of Chapter 6 .

Parallel Flow Splitter – A parallel flow splitter is where a channel of a particular width is split into two by a center pier. Each side of the pier then becomes its own channel that may direct flows into different directions.

For subcritical flow in channels, this is accomplished by doing a backwater analysis to determine the water surface elevation on both of the bifurcated channels with the desired flow rates in each channel. The channels geometries are adjusted until the headwaters in both systems match at the splitter location. The matching water surface elevation at the splitter is then used as the downstream control in the upstream channel.

If the flow in the channel at the structure site is supercritical, the process is reversed, and the water surface profiles are calculated in the downstream direction. However, considerable caution should be exercised in attempting to split supercritical flows. Readers are strongly encouraged to consult appropriate references listed at the end of this chapter or seek the advice of an experienced professional.

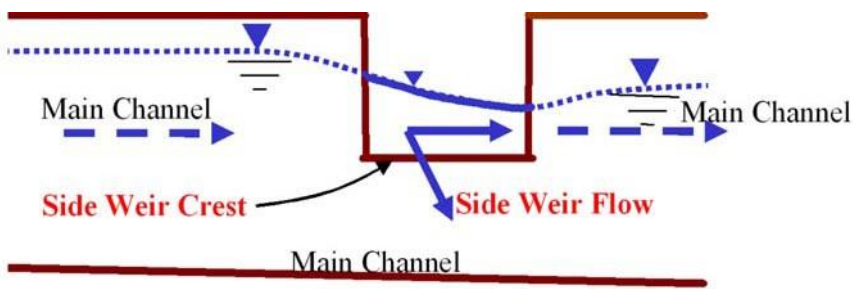
Once the water surface at the structure site has been established, the amount of flow in each area of the upstream channel can be calculated and the precise horizontal location of the splitter wall established. The initial angle of departure of the diverted channel should be minimized to reduce the formation of standing waves and turbulence that could encroach on the channel freeboard or otherwise reduce the capacity of the channel.

Side Channel Weirs – The last form of splitter is a high flow diversion, aka a Side Channel Weir. See Section 9.6 for more information.

9.6 SIDE CHANNEL WEIRS

Side weirs, also known as lateral spillways, are installed along the side of the main channel to divert water into another hydraulic structure (typically a detention basin) when the flow surface in the main channel rises above the side weir crest. Figure 9.10 shows a side view of a channel with a side weir.

Figure 9.10: Side View of a Channel with a Side Weir



Hager's Weir Discharge Coefficient Equation. Hager's equation deals with three types of side weirs: sharp-crested weir, broad-crested weir, and round-crested weir (Hager, 1987). Figure 9.11, Figure 9.12, and Figure 9.13 show the side view of these three types of weirs. In Figure 9.11, Figure 9.12, and Figure 9.13, water in the main channel flows perpendicular to the figure view.

HEC-RAS and Hager's Equation

Hager's side weir equation (Hager, 1987) has been incorporated into HEC-RAS version 4.1.0 and later for both steady state and unsteady state flows. It should be noted that if an off-line detention basin is to be designed, an unsteady state HEC-RAS model should be used. More discussion can be found in the HEC-RAS Hydraulics Reference Manual (USACE, 2016).

Figure 9.11: Sharp-Crested Weir
(Hager, 1987)

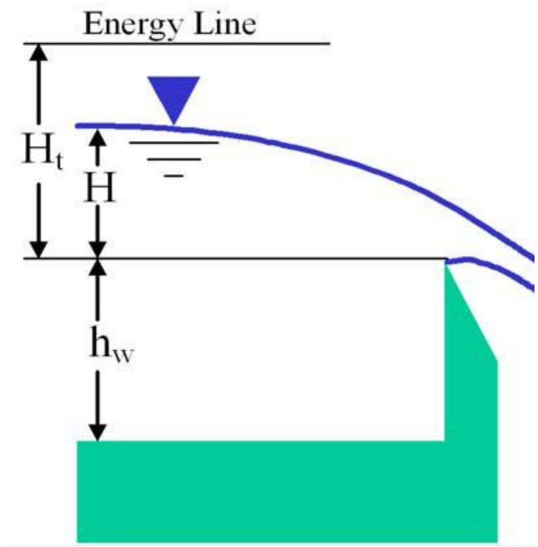


Figure 9.12: Broad-Crested Weir
(Hager, 1987)

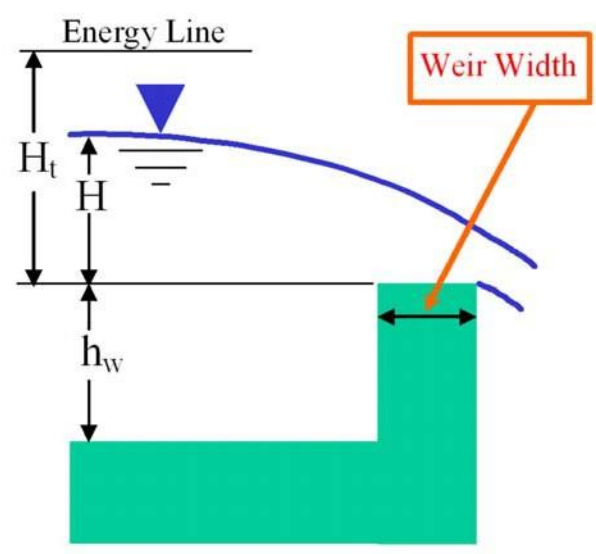
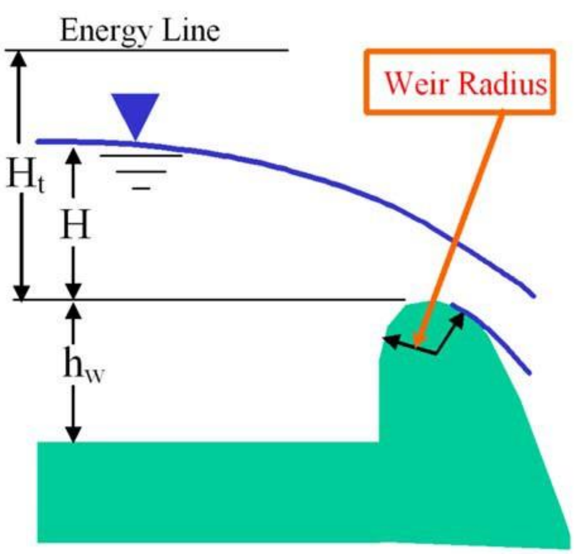


Figure 9.13: Round-Crested Weir
(Hager, 1987)



9.7 TRASH RACKS AND ACCESS BARRIERS

The necessity for trash racks depends on the size of the conduit, the nature of the trash and debris, public safety, and other factors. These factors will determine the type of trash racks and the size of the openings. If there is no danger of clogging or damage from small trash, a trash rack may consist simply of struts and beams placed to exclude only the larger trees and such floating debris. See also Section 11.3.1.

The California Standard Plans for Public Works Construction or "Greenbook" Standard Plan Nos. 360-2 and 361-2 offer details for a sloped protection barrier and typical inclined trash rack.

Trash rack losses are a function of velocity, bar thickness, bar spacing, rack angle, and orientation of the flow entering the rack, the latter condition being an important factor. Trash rack horizontal bars used to support vertically oriented bars should be as small as practical and kept to the minimum required to meet structural requirements to minimize debris clogging. Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level – the slower the approach flow, the flatter the angle.

Trash racks can promote debris buildup and the subsequent reduction of hydraulic performance. Thorough analysis of this potential should be undertaken prior to their use. To ensure that the trash rack does not limit the hydraulic performance of the associated drainage system, the following criteria shall apply:

- The Ao (open area between the bars for a specified approach depth) shall be no less than three times the cross sectional area of the associated depth through the conveyance system. For example, if a debris rack is placed in front of a 36-inch culvert flowing full, the minimum $A_o = 3 \cdot \left(\frac{\pi}{4} \cdot 3ft^2\right) = 21.2\ sf$.
- A clogging factor of no less than 50 percent shall be used for hydraulic analysis of all trash racks and access barriers. For maximum headloss, half of the net area between the bars shall be considered blocked. This will result in twice the velocity through the trash rack. For detention basin and dam outlet works analysis, trash rack headloss shall be calculated for the 50% plugged condition as well as the unplugged condition to ascertain critical scenarios for the design of downstream conveyance systems.

The trash rack/access barrier assembly shall be hinged or removable to facilitate removal of accumulated debris and sediment from around the outlet structure. If the trash rack is bolted or set in concrete, it will preclude removal of accumulated material and will eventually adversely affect the hydraulics.

The grate shall be designed to withstand the hydrostatic load resulting from the 100-year design ponding with grate openings blocked.

To calculate headlosses caused by a trash rack, use equation as indicated below:

Condition	Equation
Approach velocity ≥ 3 fps	Eqn (9.11)
Approach velocity < 3 fps	Eqn (9.12) inclined rack Eqn (9.13) vertical rack
Outlets to basin or other areas where water is ponded with low /no velocity	

Where the approaching flows are channelized or approaching with a velocity greater than or equal to 3 fps, the expected head loss from a trash rack in a channel is greatly affected by the approach angle. The head loss computed by Equation (9.11) should be multiplied by the appropriate value from Table 9.7, when the approach channel and trash rack are at an angle to each other. Equation (9.11) applies to access barriers at the upstream end of conduits and should be used when approach velocities are greater than 3 ft/sec.

$$h_g = 1.5\,\frac{[V_g^2 - V_a^2]}{2g} \times (\text{Loss Factor per Table 9.7}) \tag{9.11}$$

where:

hg = headloss through grate, (ft)
Vg = velocity of flow through the openings of the bar screen (ft/sec)
Va = approach velocity in upstream channel (ft/sec)
g = acceleration due to gravity, 32.2 ft/s^2

Table 9.7: Loss Factors for Approach Angle Skewed to Trash Rack
DERIVED FROM Metcalf and Eddy (1972)

Approach Angle (degrees)	Loss Factor
0	1.0
20	1.7
40	3.0
60	6.0

For trash racks in detention basins, reservoirs, dams, or areas where the flow into the outlet conduit is ponded, the headloss shall be computed by Equation (9.12) (Metcalf and Eddy, 1972) based on German experiments:

$$h_g = K_{g1}\,(w/x)^{4/3}\,\frac{V_u^2}{2g}\sin\theta_g \tag{9.12}$$

where:

hg = headloss through grate, (ft)
K_{g1} = bar shape factor:

2.42 – sharp edged rectangular
1.83 – rectangular bars with semicircular upstream faces
1.79 – circular bars
1.67 – rectangular bars with semicircular up- and downstream faces

w = maximum cross-sectional bar width facing the flow (in)
x = minimum clear spacing between bars (in), accounting for clogging
Vu = approach velocity (ft/s)
θg = angle of the grate with respect to the horizontal (degrees)
g = acceleration due to gravity, 32.2 ft/s^2

The Corps of Engineers (HDC, 1977) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

For vertical trash racks with low approach velocities, the following formula can also be used:

$$h_g = \frac{K_{g2} V_u^2}{2g} \tag{9.13}$$

where:

hg = headloss through grate, (ft)
K_{g2} = defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)
  $$K_{g2} = 0.00158 - 0.03217 A_r + 7.1786\, A_r^2$$
- sharp edged rectangular (length/thickness = 5)
  $$K_{g2} = -0.00731 + 0.069453 A_r + 7.0856\, A_r^2$$
- round edged rectangular (length/thickness = 10.9)
  $$K_{g2} = -0.00101 + 0.02520\, A_r + 6.0000\, A_r^2$$
- circular cross section
  $$K_{g2} = 0.00866 + 0.13589\, A_r + 6.0357\, A_r^2$$

A_r = ratio of the area of the bars to the area of the grate section
g = acceleration due to gravity, 32.2 ft/s^2

Figure 9.14: Example Trash Rack Bar Length vs. Thickness

9.8 GROINS AND GUIDE DIKES

Groins and Guide Dikes are similar structures that are installed within a natural watercourse (soft/natural bottom, but possibly bank-lined) to reduce erosion and scour at the toe of the side slope / embankment. They may also serve to train flow away from critical areas. Because of the similarity in form or function, the terminology used in practice tends to be overlapping in that the term used by one entity or organization conflicts in meaning with the same term used by another. In this Section, two hydraulic structures will be discussed. The first, identified herein as groins, are used to train flow, and reduce erosion in scenarios with engineered banks. The second, referred as guide dikes, serves a similar purpose, but are typically found in a natural floodplain setting.

9.8.1 Groins

Structures located along and protruding from the banks of an engineered channel for purposes of training flow away from the bank, reducing velocities, or reducing erosion are termed groins herein. Other terms used for structures meeting this definition are spurs, hardpoints, and dikes. Hydraulically, groins create greater depths of flow upstream of the structure in subcritical flow conditions and flatten the energy grade line. Acting like a constriction, the energy grade line is steeper at the structure while backwater eddies are created immediately downstream of the structure unless they are drowned out by overtopping flow. These structures tend to be designed to train low to moderate flows without overtopping. Higher flood flows usually overtop the structure. Under certain circumstances, groins deployed on both sides of an engineered channel can be used to flatten the energy grade line, thereby allowing a steeper channel slope. Under all applications, the appropriate hydraulic analysis should be employed to evaluate velocities under the range of conditions expected or required to meet regulatory requirements. Erosion protection is often required at the groin and downstream of the groin.

Groins may be made of many different materials including riprap, gabions, piling (wood and steel), rock and earth filled cribs. Depending upon the entity responsible for maintenance, the designer should verify acceptable materials for the application at hand.

9.8.2 Guide Dikes

These structures are deployed upstream of bridge abutments and serve to transition flow into the bridge from the floodplain. Also called guide banks, these structures have been found to minimize scour of the abutments and piers. In a natural setting, these structures are often deployed at the outside of bends in a channel to reduce bank erosion and redirect higher velocities towards the center of the channel where higher velocities are better tolerated due to armoring. In the absence of armoring, these structures merely relocate the area subject to continued erosion. Here, the scour is relocated to the head of the guide dike, thereby offering hydraulic efficiency and scour protection to the bridge structure. Design procedures for guide banks are enumerated in *Bridge Scour and Stream Instability Countermeasures* (USDOT, 2001).

9.8.3 Riprap for Groin/Guide Dike

The loose riprap d_{50} for groins, spur dike, and guide dike (Simons, Li and Associates, 1989a) is:

$$d_{50} = 0.01V_a^{2.44} \tag{9.14}$$

where:

d_{50} = the median diameter (ft)
V_a = average velocity (ft/s)

In general, the thickness of riprap for groins, spur dike, and guide dike is 1.0*d_{100} or 1.5 times d_{50} when gradation coefficient $(G) \leq 3$. When gradation coefficient $(G) > 3$, the riprap thickness is 1.5*d_{100} or 2.5*d_{50} (Simons, Li and Associates, 1989a).

9.9 SAFETY

Hydraulic structures constructed in Riverside County are typically fenced to prevent public access, however, public access may still occur. Designs for hydraulic structures must address the issue of safety for maintenance staff and the public. First, signage must be provided to identify the potential hazard of falling, flooding, or dangerous flow conditions to the public. Second, appropriate measures must be designed to keep the public away from hazardous locations. For example, adequate fencing or railings must be provided along the top of walls, such as wing walls or training walls.

Additional considerations for safety are discussed in the introduction to this manual (Chapter 1).

9.10 OPERATION AND MAINTENANCE

Hydraulic structures must be designed so they can be maintained. As with other drainage facilities, maintenance operations will consist of scheduled and unscheduled operations. Scheduled operations may include mowing, debris removal, graffiti removal, and rock replacement. Unscheduled operations are those which follow a storm event and may include debris removal, rock replacement, erosion repair, structural repairs, fence or railing repair, and other activities for which the frequency and scope cannot be predicted. Some maintenance considerations appropriate for hydraulic structures are presented below. Access to key areas for maintenance equipment and personnel is the primary consideration common to all structure types.

Slopes of 4:1 or flatter are recommended for mowing equipment on landscaped or grass bank and transition slopes. The local jurisdictional agency should be consulted regarding special circumstances for specific site constraints where a steeper slope may be necessary.

Transition areas upstream and downstream of structures must be designed to drain completely. This applies particularly to stilling basins, if allowed by the governing agency.

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10 DETENTION BASINS

10.1 INTRODUCTION

Stormwater detention (or 'route-down') basins temporarily store storm runoff and release it in a controlled manner in order to reduce or eliminate flooding or other adverse effects downstream impacts. The temporary storage of stormwater in a detention basin can often decrease the cost of downstream conveyance facilities and may also provide ancillary water quality benefits by minimizing the chance for flood water to collect pollutants as it flows through a community. Detention basins differ from debris basins (Chapter 11) in that debris basins are designed for storage and removal of sediment and debris volumes, and often do not provide detention/route-down benefits.

While new stormwater facilities (pipes, culverts, etc.) in Riverside County are typically designed to convey ultimate land use (post-developed) condition 100-year peak flows, there are cases where a 100-year detention basin may be needed to reduce that 100-year peak flow rate, principally:

- Where there is an existing downstream channel or storm drain that does not have capacity to receive and convey the full post-developed flow rate. In this case a detention basin would be needed to ensure that the capacity of that existing channel is not exceeded.
- Where the full post-developed 100-year flow rate would result in the need for an exceedingly large downstream channel. In this case a detention basin can reduce the flow rate, thereby allowing a smaller downstream conveyance system to be constructed.

This manual is focused on the above-mentioned types of detention basins, referred to herein as "regional" detention basins, which are integral to the capacity and functioning of the downstream drainage system. Such regional systems must be publicly owned and operated to ensure that the intended flood protection to the downstream communities is maintained in perpetuity.

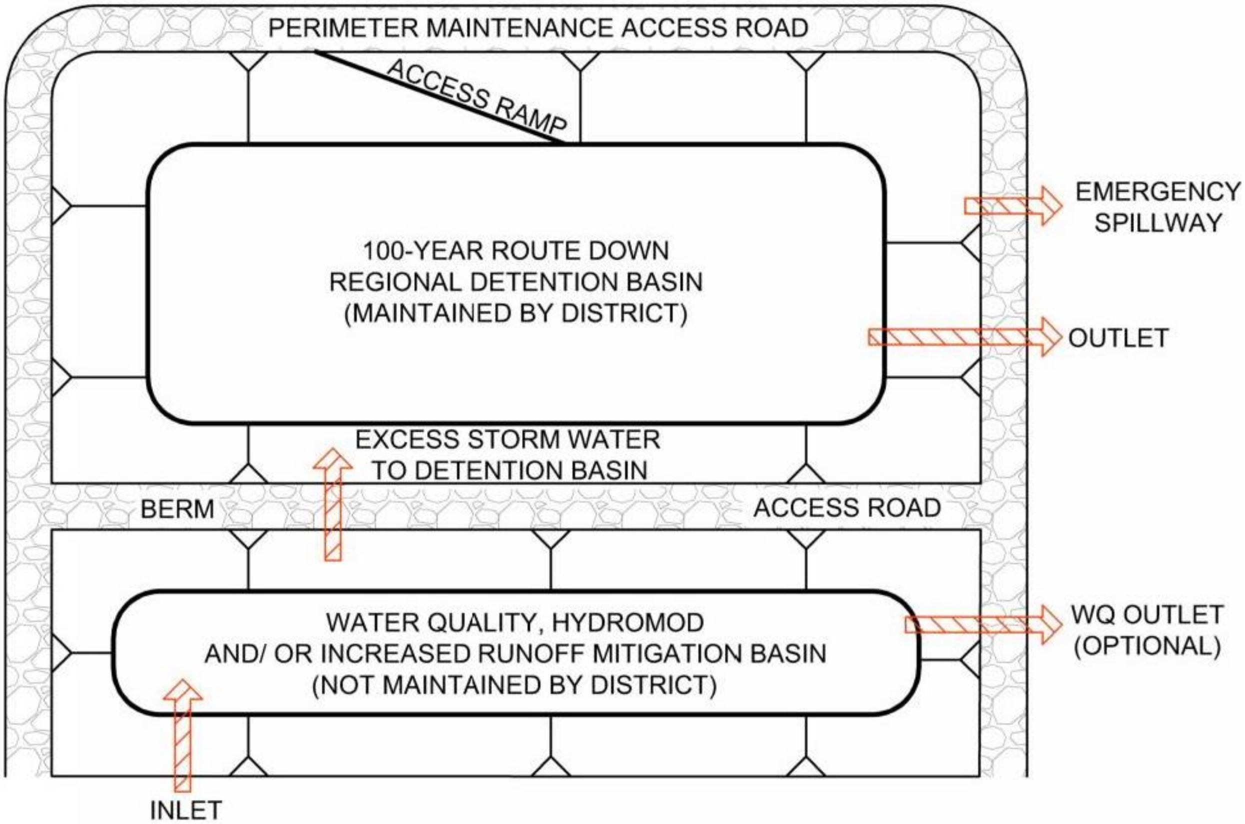
In contrast, developments frequently associated with land development projects may also need to construct localized, smaller detention basins for pollutant control, hydromodification and/or to mitigate increased runoff for lesser storm events. While the principles of detention basin routing, and many of the features described in this chapter apply and are recommended for those basins as well, these types of detention basins are not the focus of this manual, and are typically not maintained by the District and would be the responsibility of a third party to maintain. In some cases, construction of detention facilities capable of routing down flows from the 100-year peak are warranted for a single development. Due to the maintenance equipment needs and the increased risk of failure due to lack of maintenance, the District **strongly advises** that such facilities be operated and maintained by a public entity wherever feasible. The County of Riverside Transportation Department requires that detention basins in residential areas capable of routing down flows from the 100-year peak shall be maintained by a County approved maintenance mechanism. A public agency or Community Facilities District (CFD) are preferred. Industrial or commercial projects may have a Property Owners Association (POA) maintain these basins.

- For a more detailed discussion of stormwater quality issues and design criteria, the design engineer is directed to the Water Quality Management Plans and Riverside County Low Impact Development BMP Design Handbooks for the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable.
- For hydromodification requirements, refer to the Water Quality Management Plan (WQMP) Guidance Document for the corresponding watersheds, as applicable.

See Figure 10.1 for schematic of District maintained regional 100-year route-down detention basin adjacent to a conjunctive/multi-use basin that would be maintained by entity other than the District.

Note: This chapter addresses stormwater detention facilities, which only temporarily detain storm water. Stormwater *retention* facilities, also capture runoff but unlike detention facilities have no outlet structures to release water downstream. The physical configuration of a retention basin is similar to that of a detention basin, with the primary difference being that the water impounded within a retention basin is typically infiltrated for water conservation purposes, rather than discharged through an outlet works. Given that infiltration is the primary means by which retention basins are emptied of their impounded runoff, the retention basin design process is similar to design of infiltration basins as discussed in the Riverside County Low Impact Development BMP Design Handbook and, therefore, further discussion of retention basin design is not included within this Manual. Like detention basins, retention basins are only to be used with the prior permission of the governing agency.

Figure 10.1: Schematic of Region Detention Basin Adjacent to Multi-Use Basin



Stormwater detention facilities can be classified in many ways based on their design characteristics. The discussion in this manual focuses on dry-pond (i.e., no standing water between storm events), in-line stormwater detention facilities. In some cases, site constraints or other project requirements may warrant consideration of alternative types of facilities (e.g., wet pond, off-line, etc.). At a minimum, all detention facilities must meet the release rate criteria outlined in Section 10.2.1. This manual's guidance on other detention facility features, routing calculations, and hydraulic design can be adapted and applied to these alternative facilities with due care, and specific design criteria developed in consultation with the governing agency.

10.1.1 Detention Facility Categories

The following paragraphs describe several categories of detention facilities:

1. *Wet Pond versus Dry Pond.* Water is permitted to remain in wet pond detention facilities during the dry periods between storm events. This is accomplished by elevating the outlet structure above the basin bottom. For ponds to hold water over the long term, flows into the pond must exceed the loss rates from the pond due to infiltration and evaporation. Wet ponds may have aesthetic, recreational, and water quality advantages if designed and maintained properly, but can experience problems such as odors, floating debris, and vectors, such as mosquitoes, when not properly designed and maintained. Due to these concerns and the lack of perennial flows to keep such a pond from becoming stagnant, wet pond detention facilities are discouraged in Riverside County and may only be used

with prior authorization of the governing agency. Wet ponds may also have challenges with geotechnical considerations. Dry pond facilities drain completely between storm events, with outlets positioned at or below the basin bottom.

2. *At-Grade versus Below-Ground.* At-grade detention facilities (open to the sky) typically consist of a depressed or excavated area and may incorporate an earthen embankment to achieve the desired basin volume. Flood control detention basins designed to attenuate the 100-year flood event must be at-grade. Below-ground detention facilities (e.g., buried pipes, chambers, or vaults) cannot be used as 100-year route down flood control detention basin when connecting to a District maintained facility, but may be appropriate for an individual small development site, when there is not adequate surface area for above-ground detention. Use of below-ground facilities is however discouraged due to the associated maintenance difficulties. When used, below-ground facilities may have special hydraulic and maintenance requirements, such as a forebay or pre-treatment of incoming flows, that must be considered carefully during the selection and design process. Check with local jurisdiction for policies related to underground detention facilities prior to beginning the design.

3. *In-Line (Flow-Through) versus Off-Line (Flow-By).* In-line (flow through) detention basins are positioned such that runoff from the upstream watershed must pass through the basin. Only as water begins to pond up does any appreciable amount of runoff leave the basin. Off-line detention basins are positioned adjacent to and the main channel / storm drain, with runoff only entering the detention basin when the depth reaches a certain elevation within the main line. By "skimming off" only larger flows, off-line detention basins often require smaller outlet structures and less storage volume than in-line detention facilities. Due to the bypass of low flows, off-line facilities designed for flood reduction purposes are not useful for water quality treatment.

4. *Regional versus Onsite.* Regional detention basins are designed to handle flows from relatively large watershed areas. Onsite detention basins are designed to attenuate flows from a particular site.

5. *Hydraulically Independent versus Hydraulically Dependent Detention Facilities.* In some cases, detention facilities will be constructed in series. For facilities in series, they will be either hydraulically independent or hydraulically dependent. For hydraulically independent detention facilities, the hydraulic characteristics of one facility do not impact the hydraulic characteristics of the other, with the opposite being true for hydraulically dependent facilities. A common example of hydraulically dependent detention facilities in series is where the backwater from a downstream detention basin is high enough that it impacts the discharge through the outlet works of the upstream basin. Most software utilized to perform typical detention basin routing (e.g., HEC-HMS, Civil-D) does not have the capability to analyze hydraulically connected detention facilities. There are specialty software applications, such as Bentley PondPack, that do have this capability. Recognizing if a detention facility is hydraulically connected to another detention facility is

critical when designing basins in series, and both the designer and plan check engineer must be comfortable in the use of the appropriate routing software. Regarding basins in series, it is also noteworthy that design of basins in series must consider multi-day storm events in addition to the typical events being analyzed. Guidance for a multi-day analysis is provided in Section 10.2.2.5.

10.1.2 Other Basin Types

The following basin types are described for reference but are not covered in this manual.

Pollutant Control Basins: The primary purpose of water quality or 'pollutant control' basins is to enhance the quality (i.e., remove pollutants) of stormwater runoff. They are generally designed to treat runoff flow rates and volumes that are significantly less than typical flood control storm drain design events. In some cases, if the only function of the basin is for pollutant control, it will typically be designed utilizing a diversion (or splitter) structure, where low flows are split off from the main line flow and diverted to the water quality basin for treatment. In cases where the pollutant control basin also is being used as a detention basin for mitigation of increased runoff and/or hydromodification, the basin will be designed to receive the full main line flow rate, with both the mitigated flows and larger unmitigated flows being discharged from the basin through the outlet structure. Pollutant control performance standards are provided in the Water Quality Management Plan (WQMP) for the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable. Riverside County Flood Control will typically not operate or maintain pollutant control or combined pollutant control / increased runoff and/or hydromodification basins. See Section 10.5 for additional criteria regarding conjunctive/multi-use basins.

Hydromodification and Increased Runoff Basins: Hydromodification and Increased Runoff basins have the same purpose: To mitigate the impacts that converting land from natural to developed land covers can on downstream watercourses. When land develops it can change the runoff quantities and patterns, so these basins are specially designed to mimic existing or pre-development flow patterns for combinations of smaller storms such as the 2-, 5-, and 10-year events. See Section 10.4 for more information on the standards and analyses that may be required.

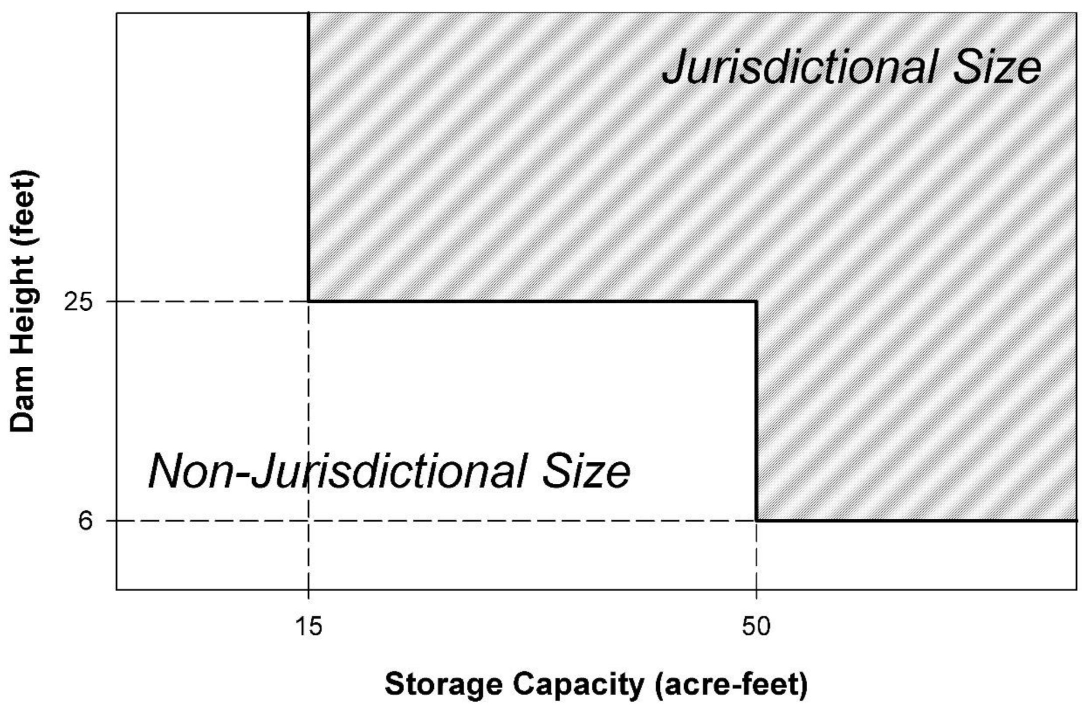
10.1.3 Jurisdictional Dams

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, or latest code).

Figure 10.2 provides an illustration of height and volume thresholds that define **jurisdictional dams** that are subject to regulation by the State Division of Safety of Dams (DSOD). If the dam height is more than 6 feet and it impounds 50 acre-feet or more of water, or if the dam is 25 feet or higher and impounds more than 15 acre-feet of water, it will be under DSOD oversight. Jurisdictional Dams require significant additional design criteria, special construction monitoring, emergency action plans and other requirements that are not covered in this manual but can add significant cost and time to a project. It is often preferable to re-design facilities such that they are not considered jurisdictional dams, if possible. The design engineer should consult with the governing agency and the maintenance entity prior to proposing a facility that they believe may be subject to DSOD oversight.

For clarity, for detention basins that do not meet the definition of being a "Jurisdictional Dam", the District recommends that it not be referred to as a dam, but rather a ‘basin’.

Figure 10.2: Jurisdictional Dam Diagram



10.2 HYDRAULIC CRITERIA

10.2.1 <u>Protection Levels / Performance Standards</u>

10.2.1.1 General

The protection goals of each type of detention facility vary. While this chapter focuses on regional detention basins that will be maintained by the District, the general design criteria provided for each type are discussed below:

1. Debris Basins: Debris basins are principally designed to store a required volume of sediment and/or debris. Hydraulically, the spillway outflow must be designed to safely pass the 100-year flow rate but may be designed to accommodate larger events as well. A low-level outlet conduit pipe is included for the purpose of dewatering the accumulated sediment and debris, however, no ‘routing’ of flows is credited. Debris basins should be sized and designed per Chapter 11 .

2. Regional (Route-Down) Detention Basins: These facilities are generally used to reduce 100-year peak flows delivered to downstream facilities. The volume of the basin will depend on the required amount of flow rate reduction; the larger the reduction, the larger volume required in the basin. The sizing of a detention basin will be an iterative process as described in Section 10.2.2.4.

   The design storm event may depend on the capacity of the downstream facility but may also be governed by other criteria. For example, a downstream natural channel may have capacity for the 100-year storm event, but in order to control the erosion within the channel, the basin may also be required to mitigate for increases in the 2-, 5-, 10-, 25-, and 50-year events. Regardless of their intended use, route-down basins are designed to accommodate the inflow, outflow, and storage of the 100-year storm event, and include an emergency spillway that is designed to pass the unmitigated (no routing credit) peak 100-year flow rate. The attenuating effects of the proposed route-down basin for the 3-hour, 6-hour, and 24-hour 100-year storm events should be analyzed. An analysis for the 1-hour duration storm event shall be provided in the event it is determined that it may govern. This can occur for basin tributary watersheds with areas approximately 500 acres or less. Should the basin take more than 24-hours to drain, a multi-day analysis of the basin is required. See Section 10.2.2.5 for multi-day analysis criteria.

Flood control detention facilities to route down 100-year storm flows are typically not required as a condition of development when the project discharges to a master-planned regional flood control facility that is designed to accommodate developed-condition flows. The design engineer shall confirm with the governing agency whether detention is required at a particular site.

The design engineer shall confirm that outflow from a detention basin shall not have a detrimental effect on downstream facilities. In some cases, detention facilities constructed with a development, while reducing peak onsite flows, might increase the peak flow from the watershed as whole by delaying the onsite peak flow rate to coincide more closely with the peak from the larger watershed. Therefore, hydrologic analyses must extend downstream from the project site an adequate distance to determine the regional effect of a detention basin.

10.2.1.2 Hydrology

Detention basins are sized using a hydrologic analysis that requires a hydrograph developed in accordance with Section E of the Hydrology Manual. Rational method peak flows are insufficient for Detention Basin design.

10.2.1.3 Drain Time

Basins shall be designed to completely drain the facility within 72 hours of the end of the storm event. Note that in certain locations where the presence of standing water may be undesirable, such as near airports due to the aviation hazard associated with waterfowl, drawdown times may be restricted to a lesser time limit. Should the basin take more than 24 hours to drain, a multi-day analysis of the basin is required.

10.2.2 Detention Routing Analysis

The design procedure for regional detention basin routing includes: 1) development of an inflow hydrograph, 2) development of a preliminary stage/storage/discharge rating curve, 3) initial routing of the hydrograph through the basin, 4) revising the basin and/or outlet configuration, and associated rating curve, based on the results of the initial routing, and 5) final routing of the hydrograph through the basin. By following the analysis procedures outlined here, the design engineer can design a flow through detention facility design that successfully meets the release rate criteria outlined in Section 10.2.1. It should be noted that the descriptions herein are tailored to use in regional detention basins, however, the underlying principles apply to smaller more localized detention facilities as well.

10.2.2.1 Inflow Hydrograph

The inflow hydrograph to a regional detention facility shall be determined using the methods outlined in the current version of the *RCFCD Hydrology Manual*. For regional detention basins or 100-year route down basins, the attenuating effects shall be analyzed for the 3-hour, 6-hour, and 24-hour 100-year storm events. An analysis for the 1-hour duration storm event shall be provided in the event it is determined that it may govern. This can occur for basin tributary watersheds with areas approximately 500 acres or less. Should the basin take more than 24-hours to drain, a multi-day analysis of the basin may be required. See Section 10.2.2.5 for multi-day analysis criteria.

10.2.2.2 Stage-Storage Curve

Stage-storage curves define the relationship between the depth of water (stage) and the storage (volume) available in the reservoir. Stage-storage curves are typically developed using topographic mapping (natural storage area) and/or grading plans (engineered storage area) for the detention facility. Examples of equations that may be used to estimate the stage-storage curve may be determined by either an average-end area calculation (Equation (10.1)) or as the volume of a conic frustum (Equation (10.2)), the latter of which is more accurate for typical detention basin configurations:

$$V_{1,2} = \frac{(A_1+A_2)}{2}(h_2 - h_1) \tag{10.1}$$

$$V_{1,2} = \frac{1}{3}\left(A_1 + A_2 + \sqrt{A_1A_2}\right)(h_2 - h_1) \tag{10.2}$$

where:

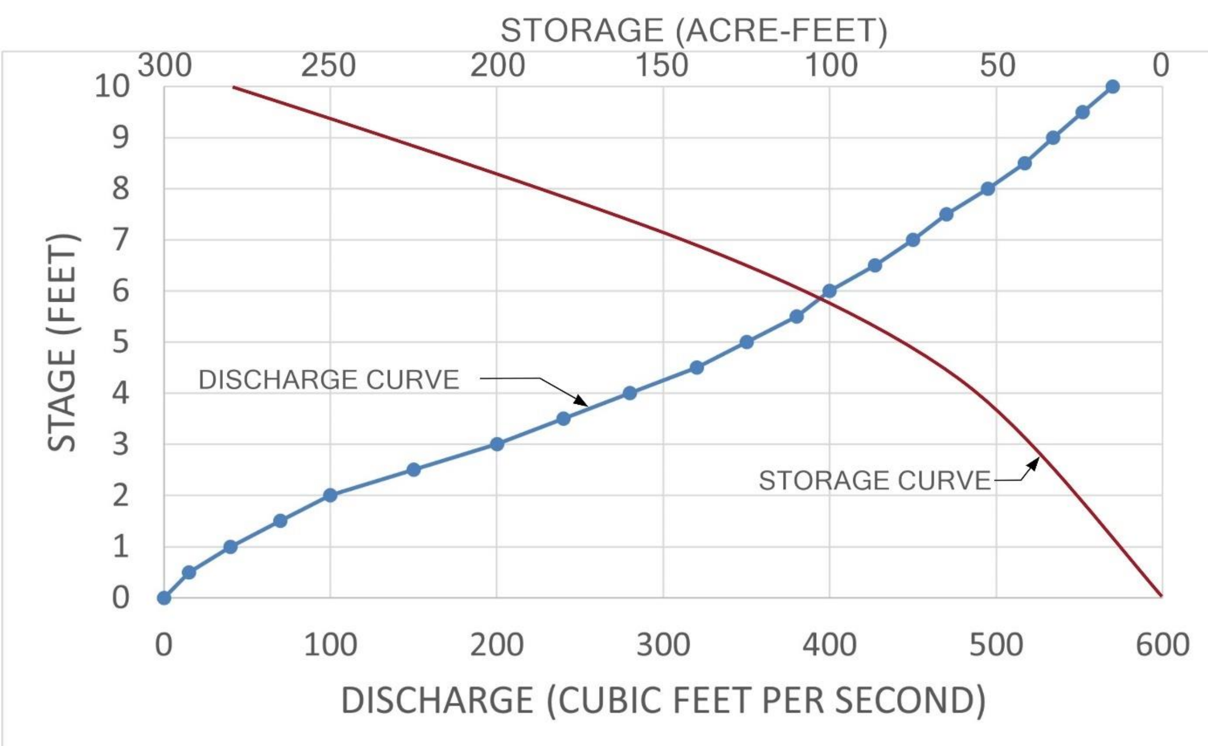
$V_{1,2}$ = storage volume between elevations h_1 and h_2 (ft^3)
A_1, A_2 = water surface area at elevations h_1 and h_2, respectively (ft^2)
h_1, h_2 = lower and upper bounding elevations, respectively (ft)

The stage-storage curve begins at the bottom of the storage basin, or the maximum elevation of sediment or debris allowed in the operation and maintenance plan, whichever is greater. Volume reduction factors may be applied to account for vegetation and/or additional sediment and debris deposition within the detention facility when necessary. For dual-use infiltration/detention facilities where the water quality volume is being infiltrated, the detention outlet opening invert will be higher in elevation than the basin bottom. In this case, utilization of the dead storage (infiltration) volume in the stage-storage rating curve may not be utilized in routing flows through a detention facility, however, infiltration of this volume must be considered when looking at the drawdown time of the basin.

10.2.2.3 Stage-Discharge Curve

Stage-discharge curves define a relationship between the depth of water in the detention facility and the outflow or release from its outlet structures. Figure 10.3 illustrates a typical stage-discharge curve.

Figure 10.3: Example of Stage-Storage-Discharge Curve



The discharge curve is developed by developing evaluating a range of outflows and associated basin depths for the low-level outlet and the spillway.

- Low Level / Primary Outlet: Follow the procedures described in Section 7.4 accounting for both Critical Depth, Orifice and Friction Control. Where a sloped basin outlet structure is required, follow the additional requirements discussed in Section 10.3.2.2.
- Spillway: Calculate the spillway flow using the appropriate weir equation as described in Section 9.4.

The final outlet rating curve for the proposed facility shall be a composite of the governing conditions analyzed above.

10.2.2.4 Storage Routing Calculations

Routing is the process of analyzing flows entering and leaving a detention facility in order to determine the change of the water surface elevation within the facility over time. Storage routing calculations are typically performed using computer programs but can also be performed with advanced use of spreadsheets. The routing of flows through a detention facility is fundamentally based on conservation of mass (Inflow-Outflow=ΔStorage), approximated as a finite-differences as:

$$\frac{S_{n+1}-S_n}{\Delta t}=\frac{I_n+I_{n+1}}{2}-\frac{O_n+O_{n+1}}{2} \qquad (10.3)$$

where:

S_n, S_{n+1} = storage within a detention facility at a time step n and $n+1$, respectively (ft^3)
Δt = time interval (sec)
I_n, I_{n+1} = inflow rate at time step n and $n+1$, respectively (ft^3/s)
O_n, O_{n+1} = outflow rate at time step n and $n+1$, respectively (ft^3/s)

The most common method for performing routing analysis for a detention facility is the Storage Indication aka the 'Modified Puls' method. The Modified Puls method rearranges the expression for mass conservation as:

$$\left(\frac{2S_{n+1}}{\Delta t}+O_{n+1}\right)=\left(\frac{2S_n}{\Delta t}-O_n\right)+(I_n+I_{n+1}) \qquad (10.4)$$

The left-hand side of Equation (9.4) is usually called the Storage Indication number. The Storage Indication method facilitates the routing analysis of detention facilities, which can be accomplished by spreadsheet calculations or using computer programs such as the Corps of Engineer's *HEC-HMS Hydrologic Modeling System (or legacy HEC-1 model)*, or proprietary software packages.

10.2.2.5 Multi-Day Storm Analysis

Should the basin take more than 24 hours to drain, a multi-day analysis of the basin may be required.

Due to the interaction of watershed size, lag, percentage of peak discharge reduction, and basin volume, the critical storm duration is generally not known (in advance) for a watershed flood control system which includes one or several detention basins. Hence, the use of the 24-hour design storm may not be the "critical" storm for flow-through detention basin design purposed, and a longer design storm may be needed.

The following supplemental design criteria shall be used for the design of basins using multi-day storm analysis:

1. The detention basin network must be set up in HEC-HMS, HEC-1, Civil-D Hydrograph Routing, or PondPack. The network is to include hydrograph development for each of the watersheds within the study area, and detention basin rating curves.

2. A constant loss rate is to be used for all storm events analyzed for detention basin in series, and District S-Graphs must be incorporated into hydrograph development per the Hydrology Manual.
3. Initially, analyze the 100-year, 3-hour, 6-hour, and 48-hour storm events. If either the 3-hour or 6-hour storm events govern for volume over the 48-hour event, the analysis stops at this point and the facilities are designed for the governing event.
4. If the 48-hour storm event governs over the 3-hour and 6-hour events, then perform an analysis of the 24-hour and 72-hour events. If the change in detention basin storage volume between the 72-hour and 48-hour events is less than the change between the 48-hour and 24-hour events, then utilize the 72-hour storm as the governing event. If the change in detention basin storage volume between the 72-hour and 48-hour events is greater than the change between the 48-hour and 24-hour events, then analyze the 96-hour event. This process must continue until the change in detention basin storage volume between two consecutive storm events is less than the change between the previous two events. When that point is reached, the storm event analyzed that has the longest duration is the governing event. As is typical in detention routing, the required detention basin storage volume will be reduced as the capacity of the basin outlet (and therefore the downstream system) increases.
5. When analyzing multiple day storm events, the daily precipitation for each consecutive day increases until the final day, where the 24-hour point rainfall depth is used. For determining rainfall depth over the first 24 hours of the 48-hour storm, the 24-hour point rainfall depth is subtracted from the 48-hour point rainfall depth, and the remaining rainfall is distributed over the first 24-hour period according to the RCFCD 24-hour rainfall pattern. In turn for the 72-hour storm, the rainfall depth for the first 24-hour period is calculated by subtracting the 48-hour point rainfall depth from the 72-hour depth, and so on. Multiday precipitation values can be obtained from NOAA Atlas II or NOAA Atlas 14 as appropriate.

10.3 DESIGN CRITERIA

All construction and materials shall confirm to all applicable specifications provided in the District's standard construction specification, or for development projects the District/Transportation Department Memorandum of Understanding at rcflood.org>Business>Engineering Tools>General>District/ Transportation MOU. The detention basin design criteria in this Section is applicable to regional above-ground 100-year route-down basin facilities to be maintained by the District, not pollutant control, hydromodification, or increased runoff mitigation basins. However, basins designed to attenuate the 100-year flood event to be maintained by the District shall have an access road around the entire basin. See Section 10.3.4 below for further access road design guidelines. For the 100-year route-down basins not to be maintained by the District, the perimeter access road is strongly recommended. For further design guidance, see Riverside County Design Handbook for Low Impact Development Best Management Practices Appendix C Basin Guidelines.

10.3.1 Inlets

10.3.1.1 Types and Hydraulics

All locations where runoff will enter the basin invert must be adequately protected. This includes major tributaries, storm drains, and both local drainage from adjacent property and from within the site (such as drainage from maintenance roads). Runoff can be introduced into the basin either through:

- Open channels (sized and designed in accordance with Chapter 8);
- Underground storm drains or culverts (sized and designed in accordance with Chapters 6 and 7 , respectively); or
- Via slope drains for small flows (designed using CH333 and an appropriate lined slope drain).

10.3.1.2 Energy Dissipation

Detention basins and debris basins shall have adequate energy dissipation and/or erosion protection at the all facility inlets to avoid damage to the basin invert, access roads, or slopes as flow enters the facility. Chapter 9 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 conveyed to the main part of the facility and are encouraged whenever practical.

Energy dissipators shall be designed assuming peak flow discharging to the basin with the basin empty. Energy dissipator structures shall be designed in accordance with Chapter 9 of this manual, utilizing the following reference documents as needed:

- 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. EM1110-2-1602, *Hydraulic Design of Reservoir Outlet Works* (Chapter 5, Latest Edition).
- An alternative approach approved by the local jurisdiction.

Under no circumstances shall an incoming stream, channel, or other inlet discharge flow onto an unprotected/unlined basin slope or invert. In addition, the review/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 the District.

10.3.1.3 Debris Barriers

Wherever possible, inlets to the basin shall be designed such that large volumes of debris will not enter the basin. Debris can reduce the capacity of the basin, and cause clogging of the basin inlet and outlet structures. Incorporating forebays and sediment traps and/or trash racks at the upstream end(s) of inlet systems can reduce the amount of sediment and debris conveyed into the basin and are encouraged whenever practical.

If a sloped protection barrier is proposed at a discharge point into the basin, the reviewer/designer should assume a minimum 50% blockage of the grate openings for determining the hydraulic adequacy of the upstream conveyance system.

For the design of debris barriers refer to Chapter 11 of this Manual or to the Los Angeles County Flood Control District, Design Manual (for) Debris Dams and Basins.

10.3.1.4 Conduit Through Embankments

Watertight joints shall be used for any conduits works placed within the basin embankment material. Such joints are not required for conduits excavated into native soils AND backfilled with CLSM in accordance with District Standard Drawing M815.

10.3.2 Outlets

Outlet for detention basins shall be designed to safely convey the design release rate that will meet intended protection levels and ensure the performance standards are met as discussed in Section 10.2.1.

10.3.2.1 Outlet Conduit

The outlet to a basin will principally be either a culvert (that is only long enough to span the embankment) or storm drain or channel system that extends for a long distance downstream. The hydraulic behavior of culverts and storm drain/channel systems is complex because of the different flow conditions (e.g., inlet control, outlet control) that must be considered depending upon the downstream conditions, flow rate, barrel geometry, and inlet opening characteristics. Chapters 6 and 8 should be followed for sizing and design of a basin outlet conduit that is a contiguous component of a longer downstream storm drain or channel system. Chapter 7 of this manual should be followed for the sizing and design of shorter outlets functioning as culvert flow. Appropriate energy dissipation shall be provided downstream of the culvert outlet works (see Chapter 9).

Watertight joints and collars shall be used for any conduits works placed within the basin embankment material. Such joints and collars are not required for conduits excavated into native soils AND backfilled with CLSM in accordance with District Standard Drawing M815.

Principal spillway outlet conduits other than those that can be analyzed by culvert hydraulics (see Chapter 7) can usually be analyzed under conditions of inlet and outlet control by procedures contained in hydraulic references such as *Design of Small Dams* (USBR, 1987, pages 453-470) or Brater and King (1976). It is important to note that such structures must be analyzed for both inlet and outlet control with appropriate consideration of tailwater conditions that may exist at the outlet of structure.

10.3.2.2 Trash Racks and Debris Control

Trash racks and debris control structures can be crucial to the successful operation of a detention facility, especially at its outlet works. See Section 11.3.1 for additional guidance regarding trash racks. In District-maintained detention basins, a sloped basin outlet structure per Section 10.3.2.3 is required.

10.3.2.3 Sloped Basin Outlet Structure

The District requires a sloped basin outlet structure to be installed in front of outlets to all District-maintained detention basins that may receive debris loads of any nature. Most basins will require this structure to facilitate continued operation during high debris flow events. Consult with the District prior to proposing a basin outlet that does not use this structure. The structure is shown conceptually in Figure 10.4 and includes a slotted grate system that extends to the elevation of the basin spillway.

Note: The District is in the process of developing a new standard drawing for sloped basin outlet structures. Check the District's website https://rcflood.org/engineering-tools for updates for this pending standard drawing. Any requirements in that published standard drawing will supersede any conflicting requirements in this Section.

Until the above mentioned standard drawing is published, the sloped basin outlet structure design must mimic the designs for Mabey Canyon (Dwg. No. 2-0438), Tahquitz Creek (Dwg. No. 6-0251), or Oak Street (Dwg. No. 2-0162) debris basins. See Section 10.7 Appendix A for example plans.

Tower type outlet structures are prohibited on District-maintained facilities.

Basic components of this outlet structure include:

1. Grate Design – Slotted grates are installed along the interior slope extending from the basin floor to the spillway elevation. The grates should be designed considering the clogging and Ao requirements described in Section 9.7.

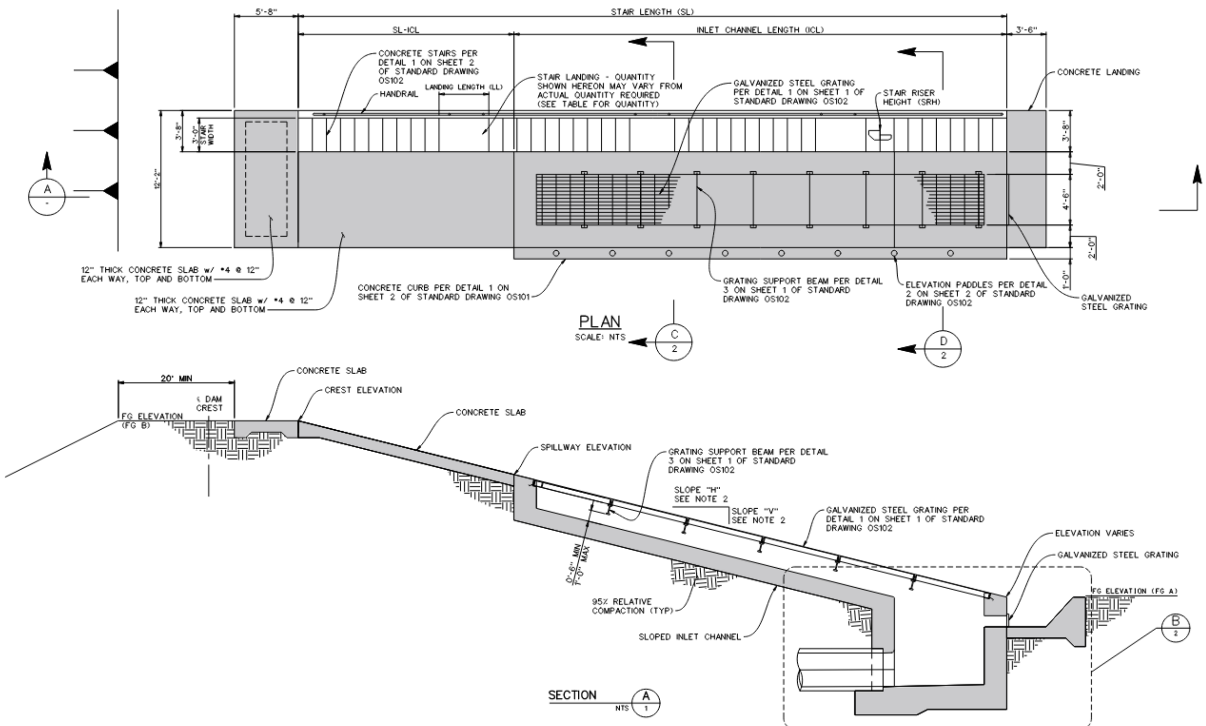
2. Free Discharge – Basins must be designed to freely discharge without gates or other manual control mechanisms to preclude standing water within the basin. Existing basins/dams that rely on gates may continue to be designed with gate-controlled discharge in close coordination with the District.

3. Maintenance Access – Access must be designed and incorporated along the entire length of the sloped grate channel for both heavy equipment and personnel. Personnel access shall include concrete stairs located adjacent to the outlet structure that extend from basin invert to the top of slope and shall include safety hand rails. Outlet structure headwalls shall contain cable fencing.

4. Visible depth staff gauges shall be provided along the slope at 5-foot vertical increments along the sloped outlet structure. See example staff gauges used in Seven Oaks Dam in Section 10.7 Appendix A. **Note: The District is in the process of developing a new standard drawing for depth staff gauges. Check the District website https://rcflood.org/engineering-tools for updates for this pending standard drawing.**

5. Minimum design loading for the outlet structure shall meet earth, surcharge, earthquake, equipment, and maximum hydrostatic loading (based on the design depth).
   - Minimum assumed allowable bearing capacity shall be 1,500 pounds per square feet.
   - Equipment live loads shall be 14,000 pounds.
   - Moist and saturated unit weight of 120 pounds per cubic feet can be used to estimate equivalent fluid pressures.

Figure 10.4: Sloped Basin Outlet Structure



Due to the specific geometries of this outlet structure and the fact that the outlet conduit is depressed below the basin floor, the methods for determining the stage-discharge curve (see Section 10.2.2.3) must be modified as follows:

- Critical Depth Control does not apply. Instead analyze weir flow through the grate as described in Section 9.4.2. When determining the weir length, accounting for the vertical bars and clogging. As depth in the basin increases, the effective height of the weir increases. At some point the structure outflow will become limited by either Orifice Control or Friction Control.
- For Orifice Control, ensure that the calculated height above centroid of the conduit is reconciled to the basin floor. For example, a 1-foot depth in the basin may (depending on geometries) be associated with a depth above centroid of the conduit of 4 feet.

Headlosses through this structure can be calculated per Equation (9.12) using a sharp edged rectangular bar shape factor.

Once water flows into the structure through the grates, the other components of the outlet structure, including the sloped channel, gate support beams, etc. must be designed in such a manner that they will not be the limiting factor at any stage. For example, the hydraulic capacity of the sloped outlet channel shall be evaluated to confirm the discharge capacity is equivalent or greater than the discharge capacity of the basin.

10.3.2.4 Retrofitting Existing Dams or Basins

Existing dam and large basin facilities often undergo retrofit or modification of outlet structures at end of service life or to improve maintainability of the outlet. These rehabilitation projects generally consist of replacing deficient or damaged structural elements to not only extend the life of the facility but also reduce maintenance intervals. Where outlet structures are being replaced, the replaced structure should follow the requirements of this chapter.

If a new outlet structure is being implemented into an existing dam or basin facility, the as-built or record drawings shall be analyzed. Specifically, the as-built outlet rating curve should be reviewed in comparison to the current baseline or existing condition. Any post-construction modifications, improvements, or otherwise shall be accounted for to establish an existing condition outlet rating curve for comparison to the as-built condition. Examples of common modifications include outlet pipe extensions, pipe resurfacing/restoration, spillway modifications, and the installation of outlet structure grates. These outlet rating curves are developed by evaluating the various hydraulic conditions outlined in Section 9.7 and are modeled to the spillway elevation. These as-built and baseline outlet rating curves will be compared with the proposed improvements. The following hydraulic analyses must be performed:

A. As-Built Outlet Rating (Stage-Discharge) Curve and Baseline Outlet Rating Curve

B. Proposed Outlet Rating (Stage-Discharge) Curve

C. Proposed Outlet Structure hydraulic analysis

D. Proposed air ventilation pipe size calculations

10.3.2.5 Air Ventilation Pipe Sizing

Air ventilation pipes are commonly used to increase pressure downstream of a slide gate on gated dam facilities. These ventilation pipes reduce the risk of cavitation/avoid vacuum from occurring in the outlet pipes and are recommended to be a minimum of 4" in diameter. Installation of a mesh screen on the inlet of the ventilation pipe shall be provided to preclude large debris from entering the ventilation pipe. The Engineer is required to calculate the air ventilation pipe size utilizing the methodology provided in "Small and Medium Dam Air Vent Sizing for Low-Level Outlet Works," by Tullis, Blake P. and Larchar, Jason. World Environmental and Water Resources Congress 2010. The air ventilation pipe diameter shall be clearly indicated on the plans.

All calculations and corresponding backup shall be memorialized in report format, stamped, and signed by a registered civil engineer.

10.3.2.6 Riser Structures

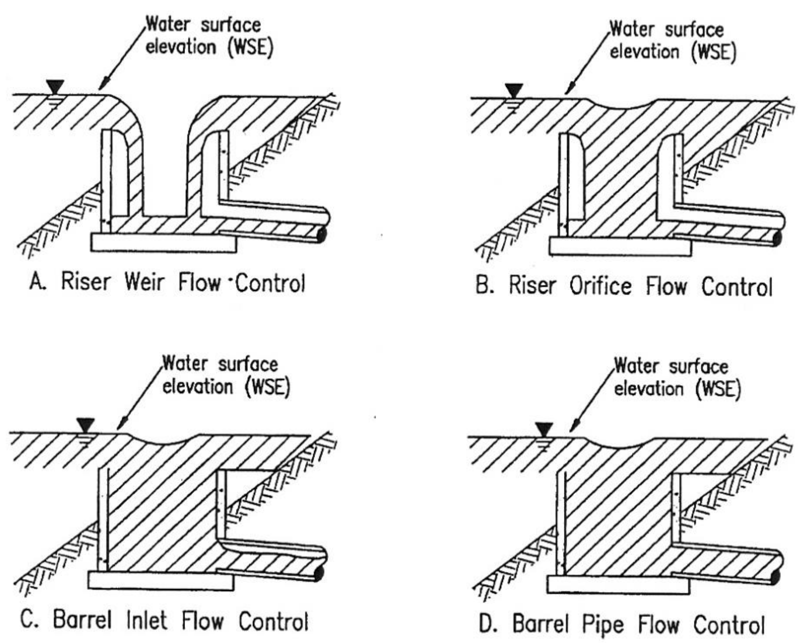
The District typically does not allow riser structures for the principal (low-level) outlet conduit for regional detention basins or debris basins. The discussion below is provided for refence and use in other types of basins or situations.

Riser structure is a general term for structures having inlet openings that are parallel to the water surface in a detention facility. Riser structures with circular cross-section are often called standpipes, and rectangular riser structures are often called inlet boxes. Figure 10.5 illustrates the hydraulic behavior of a typical riser structure. 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; and
4) Barrel pipe flow control.

The USBR *Design of Small Dams* (1987) discusses the hydraulics of riser structures in more detail.

Figure 10.5: Hydraulic Control Through a Typical Riser Structure



When the water surface reaches the top edge of the riser, flow will typically begin to pass over the edge of the structure in the manner of a sharp-crested weir (for sharp-crested weir equation and coefficients, see Section 9.4), 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. The horizontal orifice flow depends upon the area of the top of the riser structure and can be computed using the following equation:

$$Q = C_{HO} A_O \sqrt{2g(h - h_R)} \tag{10.5}$$

where:

Q = orifice flow discharge (ft^3/s)
C_{HO} = horizontal orifice discharge coefficient (dimensionless)
A_O = cross-sectional area of flow through the orifice (ft^2)
g = gravitational acceleration (32.2 ft/s^2)
h = head elevation above the riser orifice (ft)
h_R = elevation of the crest of the riser orifice (ft)

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 (h_T) to simplify analysis. The transition water surface elevation ($h=h_T$) is found by calculating the point at which the weir equation and orifice equation yield the same discharge:

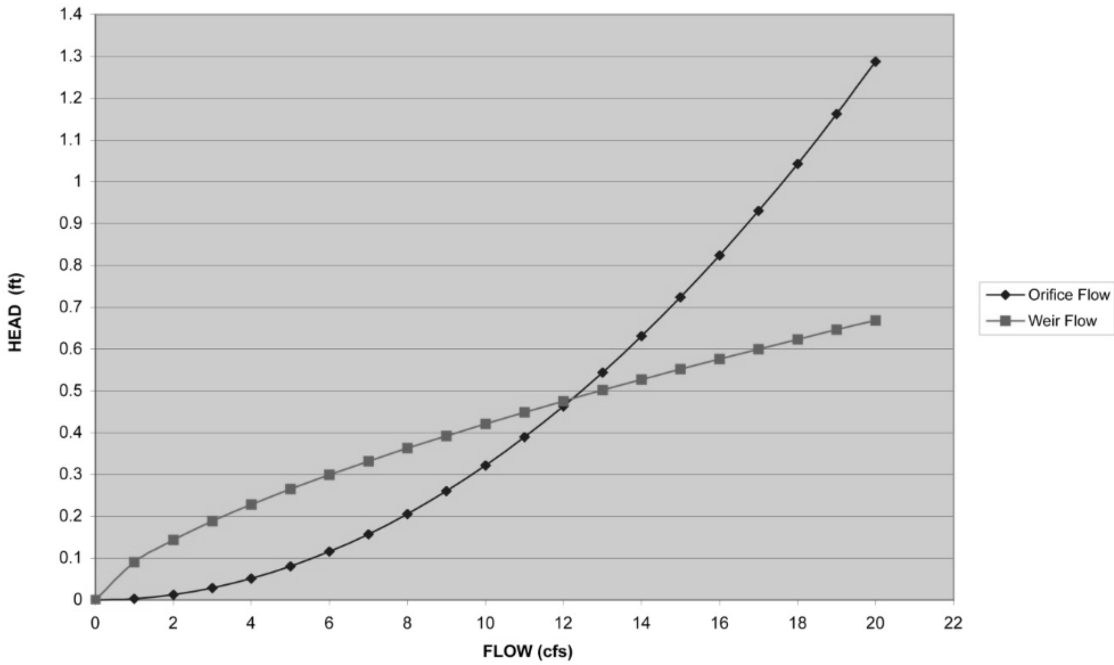
$$h_T = h_c + \frac{C_{HO} A_O}{C_{SCW} L} \tag{10.6}$$

where:

h_T = discrete water surface elevation (ft)
h_C = discrete water surface elevation (ft)
C_{HO} = horizontal orifice coefficient (dimensionless)
A_O = cross-sectional area of flow through the orifice (ft^2)
C_{SCW} = sharp-crested weir coefficient
L = length of weir crest (ft)

Typically, this analysis is computed using a spreadsheet to calculate the flow rates at incremental depths for both the orifice and weir equations as shown in Figure 10.6. The lower flow rate is the governing flow rate.

Figure 10.6: Orifice vs. Weir Flow for Riser Structures



Thus, the weir equation is used for calculating flow through a riser structure for water surface elevations $h \leq h_T$ and the orifice equation for water surface elevations $h \geq h_T$.

As the water surface elevation rises further, the control can change to barrel inlet flow control and/or barrel pipe flow control.

10.3.2.7 Perforated Risers

The District typically does not allow riser structures for the principal (low-level) outlet conduit for regional detention basins or debris basins. The discussion below is provided for reference and use in other types of basins or situations.

Perforated risers typically incorporate small openings on the sides of the riser, which are used to regulate discharge from a detention basin for storm events smaller than the basin design storm. As such, they are often useful in water quality treatment applications or to mitigate increased runoff from lesser storm events (e.g., 1-year, 2-year, etc.). 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 riser.

Within basins, the District does not maintain riser structures.

Basin 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.

For any basins that will be maintained by the District, including regional detention basins, the following criteria apply:

Outlet structures with an inside dimension of 36 inches or larger and a vertical interior height greater than 6 feet shall have ladder rungs or similar safety devices to facilitate access by maintenance personnel. See Section 10.3.2.9 for buoyancy issues. Additional means of access may be required by the governing agency.

Riser pipes within basins will not be maintained by the District.

10.3.2.8 Multi-Stage Outlets

Combinations of culverts, weirs, orifices, and riser structures can provide multiple-stage outlet control to mitigate increases in developed condition discharge for different storm frequencies, as well as to mitigate water quality impacts by metering discharge from a basin for a given storm volume. These combination outlets may have separate independent outlet controls, but often will be part of a common outlet structure. 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 outlet conduits tailwater conditions. Figure 10.3 illustrates a typical stage-discharge curve for a detention facility outlet works.

10.3.2.9 Buoyancy

Buoyant forces create uplift forces that can damage detention basin riser structures. Outlet structures shall be anchored properly such that they will withstand buoyant forces. This is of particular concern with corrugated metal standpipes, where the weight of the standpipe can be far less than the buoyant force acting on it. 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 (see Section 6.2.11 for buoyancy calculation conditions).

10.3.3 Spillways

Overflow spillways are required for all basins, to control the path flows will take if the basin overtops, and to protect the basin (including any embankments) and adjacent properties and infrastructure. If the basin is entirely ‘incised’ below adjacent grade, the spillway may simply be a depressed section of the perimeter access road that is paved with concrete as described below. In basins with embankments, the spillway may be a more significant structure. 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 "undetained" 100-year design event (i.e., the maximum 100-year peak flow that enters the basin).

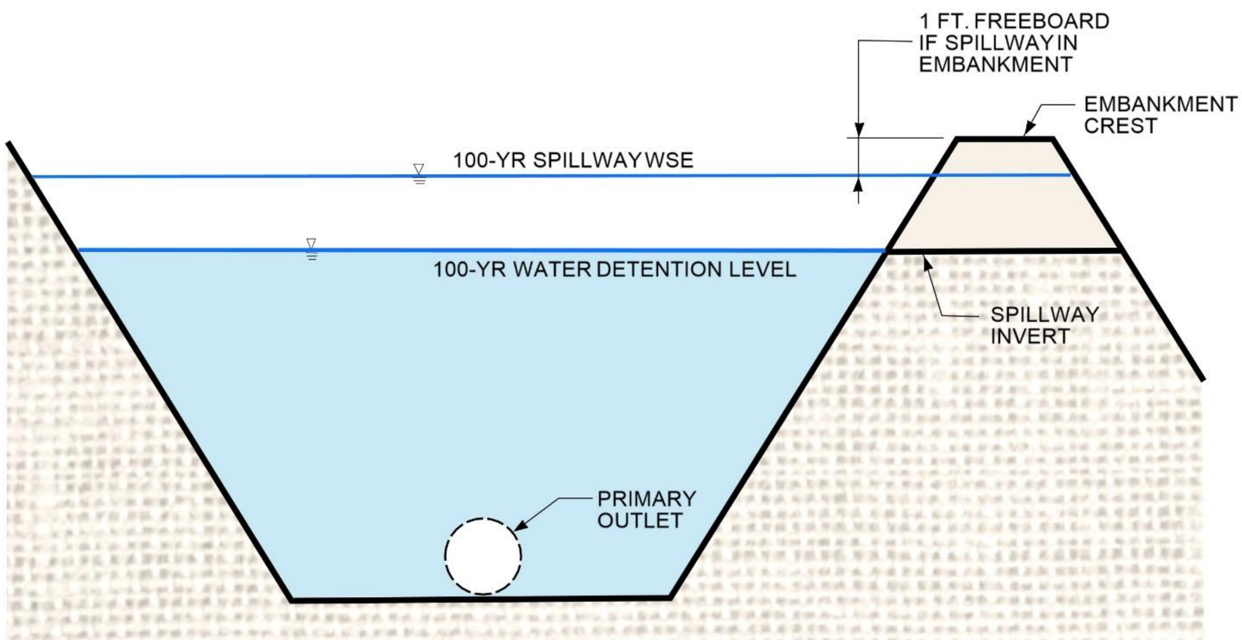
The guidance herein doesn't cover spillways that may be associated with a jurisdictional dam (see Section 10.1.3). Dam spillways require additional hydraulic and structural considerations that are beyond the scope of this manual.

10.3.3.1 Hydraulic Sizing

Detention basin spillways shall be sized to pass the 100-year flow assuming primary outlet structure is blocked, see Figure 10.7. In addition, where one or more basin side slopes are in an embankment condition, a minimum of one foot of freeboard shall be provided for the spillway (i.e., 100-year water surface elevation at least one foot below top of embankment). Additional spillway capacity and freeboard may be required for basins that meet Division of Safety of Dams jurisdictional criteria.

The flow over the spillway will be analyzed as a weir in accordance with the methods presented in Chapter 9 . Most detention basin spillways would be characterized as broad crested weir, but depending on the specific design, other forms of weir formula may be appropriate.

Figure 10.7: Detention Basin Spillway Example



10.3.3.2 Spillway Lining

Spillway lining shall consist of a minimum of 6-inch thick Class "A" Concrete with #4 at 18" O.C. reinforcing steel and 2-foot cutoff walls on the entire perimeter of the spillway per District Std. CH326. Construction of spillways shall meet all appliable specifications provided in the District/Transportation MOU specifications.

(rcflood.org>Business>Engineering Tools>General>District/ Transportation MOU).

Energy dissipation shall be provided at the outside toe of spillway where spillway is in an embankment condition. Otherwise, erosion control measures shall be provided.

10.3.3.3 Spillway Bollards

Where a basin has the potential to receive floating debris, the engineer should consider whether bollards should be installed to trap floating debris within the basin and prevent such material from being conveyed over the spillway, potentially causing downstream impacts. See Section 11.3.2 for more information.

10.3.3.4 Downstream of the Spillway

Emergency escape flow path for any water over the spillway shall be determined. To prevent flood damage to proposed and existing structures, all structures along the emergency escape path shall be protected from flooding by either properly elevating the finished floor in relation to the flow path, or by making sure the structures are setback to provide adequate flow though area in the event the emergency escape of the stormwater runoff is necessary.

Energy dissipation shall be provided at the outside toe of spillway where spillway is in an embankment condition. Otherwise, erosion control measures shall be provided.

10.3.4 Side Slopes

1. Interior and exterior basin slopes shall be 3:1 or flatter.
    - If there are secondary or recreational uses intended, additional constraints may be required by others such as fences, milder slopes, or surfacing.
2. The geotechnical report must specifically address and support basin design, including at minimum the following:
    - Stability of cut/fill and embankment slopes under both in-situ and saturated conditions that can result from the basin-full condition
    - Embankment material requirements and suitability of onsite soils

10.3.5 Basin Floor

1. Minimum bottom footprint of a basin shall be sufficient for the invert maintenance roads described in Section 10.3.11.
2. The basin floor must have positive drainage, be sloped at minimum 2% toward the outlet, and 2% laterally toward the low flow if one is provided. This requirement does not apply to retention basins.
3. A low-flow channel, if required, shall not be placed adjacent to embankments. The low flow channel may either be unlined or lined depending on whether incidental infiltration is desirable. Consult with the maintenance entity to confirm whether a low flow lining is required. If concrete-lined, the lining should be constructed per District Standard Drawing CH333.

10.3.5.1 Dead Storage

Dead storage is created when the outlet to the basin is set higher than the lowest point within the basin floor, creating a zone that will not drain except through infiltration. Dead storage is generally not allowable in detention basins except in specific scenarios and with approval of the Engineering Authority and maintenance entity. The volume of the 'dead storage' zone, when used, is typically to provide for some sedimentation capture volume, and as such must not be counted toward the stage-storage in the routing calculations.

10.3.6 Embankment

1. Embankments up to 3 feet in height (as measured from the design water surface for the spillway event to the lowest downstream toe of embankment material) may be constructed in accordance with the applicable specifications in the District/Transportation Department MOU (rcflood.org>Business>Engineering Tools>General>District/ Transportation MOU).
2. Embankments more than 3 feet in height will necessitate individual District review and additional requirements on a case-by-case basis.
3. Embankments that exceed thresholds for jurisdiction by the California Department of Water Resources - Division of Safety of Dams will need to be approved by both the District and the Division of Safety of Dams (see Section 10.1.3).

10.3.7 Vegetation

Landscaping requiring irrigation is prohibited within debris basins designed for District ownership and maintenance unless it is **installed, managed, irrigated, and maintained through other approved maintenance entities** via an appropriate license agreement. Consult with the District prior to proposing or relying on any concepts that include such landscaping.

Hydroseeding as necessary for site and slope stabilization may be acceptable provided that it 1) not require irrigation, and 2) is a mix approved by the District, and 3) not be part of any mitigation requirements that require ongoing or recurrent maintenance or re-establishment.

10.3.8 Right of Way Requirements

The project must set aside sufficient land for the full operation and maintenance of the basin, including site access roads, maintenance roads, inlets, outlets, spillways, etc. The project owner and design engineer shall consult the governing agency for determination of the appropriate maintenance entity for a particular project. When detention facilities will be privately owned and maintained, they shall have a recorded easement binding the maintenance responsibility on successors, or another similar mechanism acceptable to the governing agency.

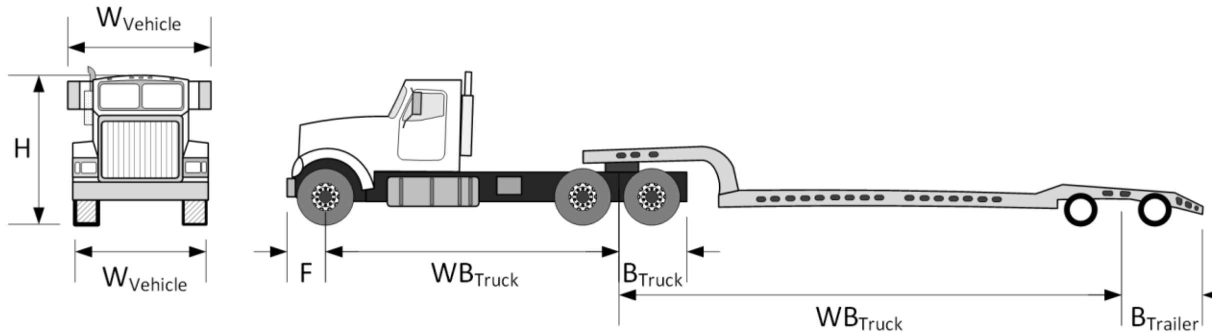
Right of way for District maintained basins must be obtained/provided in fee unless use of an easement is approved in writing by the District before environmental permitting. Easements for District-maintained basins are typically only allowed when another public entity will be responsible for owning and maintaining the site for multi-use, such as for a park.

The surface flow path downstream of the spillway must be within public right of way (such as a road), or a secured flooding easement if across private property.

10.3.9 Site Access

1. Unobstructed access from a public right of way shall be provided to meet the following vehicle design limits:
   - **Truck and Trailer** setup equivalent to or better than the RCFCWCD Standard Truck and Trailer defined in Figure 10.8.
2. Internal access roads shall be a minimum of 15-feet wide.
3. The access to the site shall accommodate the specified **Truck and Trailer** defined above for offloading of equipment within District right of way.
   - If a public street is either too narrow, consists of a single lane in each direction, or contains a raised median, then additional right of way may be required for District vehicles to safely maneuver onto travel ways from basin site.
   - In certain situations, an extended width driveway which accommodates the loading and unloading of equipment parallel to the roadway may be used. See Sheet 6 of 12 from Heacock Channel, Stage 4, Drawing No. 4-1088 in Section 10.7 Appendix A for an example.
4. Commercial size curb cut-outs and driveway approaches equal or better than RCTD Std. Dwg. 207A shall be provided.

Figure 10.8: Standard RCFCWCD Maintenance Truck Trailer



Dimension	Description	Value
W_{Wheel}	**Wheel Outside Width**	8.0-feet
$W_{Vehicle}$	**Truck Overall Width**	8.5-feet
H	**Truck Height**	9.0-feet
F	**Distance from front axle to front of truck**	3.0-feet

Dimension	Description	Value
WB_{Truck}	**Truck Wheel Base**	18.0-feet
B_{Truck}	**Distance from rear pivot point to the back of truck.**	5.0-feet
$WB_{Trailer}$	**Trailer wheel base**	36.0-feet
$B_{Trailer}$	**Distance from trailer rear pivot to back of trailer.**	7.0-feet
	Max wheel turning angle	30°

10.3.10 Perimeter Fencing

1. Perimeter fencing consisting of 6-foot chain link fence and 14-foot double drive gates per District Standard Drawing M801 shall be provided.
   - Gates, bollards, or other features shall not conflict with site access requirements listed above.
2. Perimeter fencing across streambeds shall consist of cable or barbed wire fencing per District Standard Drawing M826 or M818, or utilize a three-wire fence per Caltrans Standard B11-47.
3. Alternatives must meet or exceed these minimum criteria. If the basin is proposed for District Maintenance, alternative perimeter fencing must be pre-approved by the District and may require a separate maintenance entity to take responsibility for the fence/wall.

10.3.11 Maintenance Access Roads

1. Location: Maintenance roads shall extend around the entire perimeter of the basin along the top of the basin slope / embankment and be fully accessible during a design level storm event. At a dead end where a continuous road is not feasible, a turnaround per District Standard Drawing M827 shall be provided.

- For an embankment condition, in addition to the maintenance road at the top of the embankment, a standard access road shall be provided along the land side toe (dry side of an embankment) for maintenance operations.

2. Geometry: Maintenance road geometries must meet the following criteria:
   - Minimum 20-feet wide
   - Minimum inside turning radius of 50 feet
   - Maximum longitudinal slope of 10%
3. Surfacing: Surfacing for maintenance roads, turn arounds, and other areas within District right of way as shown on the project drawings and as directed by the Engineer shall be Filter Material in accordance with the District/Transportation MOU. Rock shall not contain recycled concrete products. The subgrade for the road must be 95% relative compaction and graded clean before placement of the surfacing.
4. Surface flow run-on to access roads shall be avoided. Where infeasible to avoid run-on, drainage aprons per District Standard Drawing CH333 shall be provided with appropriate down drains to protect the slope of the basin.

10.3.12 Invert Maintenance Access

1. At least one continuous concrete paved invert access ramp (minimal concrete compressive strength of 4000 psi) shall be provided from the top to the invert of the basin. Larger basins may require multiple ramps. All concrete access ramps shall be applied with a heavy broom finish for maintenance and emergency vehicle tire traction during severe weather conditions.
2. Invert access ramp geometries must meet the following criteria:
   - Minimum 12-feet wide and maximum 10% slope per District Standard Drawing CH330, CH331, CH334, or CH335.
3. An invert maintenance road meeting the requirements of this Section must be provided along the basin invert, connecting the invert access ramp to the outlet structure. The invert maintenance road should be elevated at least 1 foot above the adjacent basin invert grade.
4. A vehicular turn around area per RCFC Standard Drawing M827 and with a maximum cross slope of 2 percent (2%) shall be constructed at or near the terminus of the maintenance road. Where inlet flows drain across the required invert maintenance road, a drainage apron per District Standard Drawing CH333 shall be used.

10.3.13 Detention Basin Plans

Plans for the detention facility shall be prepared on separate sheets with the agency responsible for maintenance noted on drawings.

The design engineer shall include the following on the basin drawings:

- Inflow: Maximum flow rate and velocity, with water surface profile plotted;
- Outflow: Maximum total flow rate and velocity from each component of the outlet works, with water surface profile for each plotted;
- Volume: Design storage volume as measured to the emergency spillway flowline. Include a single graph that displays depth-storage-discharge relationship for the basin, including depths beginning at the lowest elevation of the basin through the capacity of the emergency spillway. Depths shall be in feet, storage shall be in acre-feet, and discharge shall be the total discharge from the basin in cubic feet per second. A stage-storage-discharge curve must be plotted on the plans;
- Details: Include details with sizes and project-specific elevations for the facility inlet, outlet structures, energy dissipaters, emergency spillway, ramps, access roads, sloped protection barrier, etc.; and
- Cross Sections: Cross-sections through the facility in at least two (orthogonal) directions. Sections must show the inflow path, outlet works, emergency spillway, as well as other sections as needed to confirm ponding depths within the basin and as needed for construction.

10.3.14 Operation and Maintenance Plan

The Operation and Maintenance Plan for most regional at grade route-down detention facilities will be covered by the RCFCD General Procedures for the Operation and Maintenance of Facilities Manual. All detention facilities require maintenance to ensure proper function throughout their lifetime. Some basins will require a project specific operation and maintenance plan sheet to be developed and included in the as-built drawing set. The operation and maintenance plan shall provide direction / instruction to the maintenance entity to ensure that they know how to preserve the function of the facility in perpetuity. The plan sheet shall specify:

- Regular inspection and maintenance at specific time intervals (e.g., annually before the wet season, after storms, etc.); not required for District-owned basins.
- Identify/Highlight all structures that must be maintained to ensure the proper functioning of the basin, to ensure that over time and with staffing changes, critical elements of the facility are not lost ore removed.
- Maintenance ‘indicators" when maintenance will be triggered (e.g., presence of vegetation, an accumulation of 6 inches of sediment and debris, or the basin does not drain within 72 hours).

Locations or areas of sensitivity, if any, where maintenance may be limited, such as underground/overhead utilities, habitat corridors, limitations on times of year, etc. shall be noted. Operation and maintenance plans shall ensure that vegetation is removed or maintained on a regular basis to preserve the function of the facility. Additional maintenance procedures may be required on a case-by-case basis.

Non-stormwater discharges shall be mitigated outside of District right of way such that they do not produce nuisance issues within a District owned basin.

10.3.15 Environmental Permitting for Maintenance

Detention facilities are often located within or adjacent to sensitive environmental areas. The design engineer must investigate which permits might be necessary from the various regulatory 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 State Water Resources Control Board

It is important that final permits and/or permit conditions allow for the perpetual maintenance of a detention facility without the need of returning to the permitting Agency. Mitigation within basins to be maintained by the District is not permitted.

10.3.16 Safety

Although many regional detention basins will be fenced off from the public, all basins should be designed considering the potential for unintended access, and for maintenance personnel safety.

In some instances, high flood flows may be directed into a multiple-use stormwater detention basins by means of a storm drain, open channel, an overflow side channel spillway, or a drop structure. A large volume of water entering the basin at high velocity can carry away an individual who is on or near the inlet structure. The design of an inlet that minimizes the velocity of incoming water will greatly enhance safety and must be included in the criteria for inlet structure design.

Within a stormwater storage facility, safety concerns increase with an increase in potential water depth. A facility with a design water depth of 2 to 3 feet (less than the head height of most users) is typically less dangerous than a facility with a design water depth of 5 to 6 feet or more.

In addition to slopes, consideration should be given to bottom conditions in flood-prone areas. Soils that provide firm footing when saturated are safer than soils that do not. In cases of severely unsuitable soils, partial or total removal may be necessary to provide for safety of the public and maintenance personnel.

Outlet conduits are an attractive nuisance during dry weather and can be a significant hazard in wet weather. Outlet structures shall be designed to minimize the likelihood of a person entering (intentionally or unintentionally) and becoming trapped during wet or dry conditions.

A properly designed trash rack can not only prevent clogging by debris but also prevent a person from being swept into the stormwater management facility. It is important to note, however, that even an outlet structure fitted with a trash rack is not safe during flood conditions. Swift moving water can submerge a person at the outlet structure regardless of the existence of a trash rack or grate. Signage is important to alert the public of this danger.

Railing or fencing shall be provided at the top of all walls and steep slopes, and stairs shall be provided adjacent to the outlet structure to reach the top of the basin.

All site furnishings, such as benches, trash receptacles, and picnic tables, should be secured to prevent them from becoming waterborne debris that could clog the outlet structure.

Safety should also be considered downstream of outlet structures. Specific conditions downstream of an outlet should be evaluated in terms of safety. To protect the public, structural walls should have fencing or railing along the top of an outlet structure.

Appropriate signage warning that areas are subject to flooding during storm events shall be provided for detention facilities, particularly when designed for conjunctive recreational use. Detention facilities located near roadways shall be guardrails or other safety measures acceptable to the governing agency.

10.3.16.1 Signage

Appropriate signage warning that areas are subject to flooding during storm events shall be provided for detention facilities designed for conjunctive recreational use. For non-recreational uses the District's standard signage warning against trespassing shall be used. Other signage may be necessary as well to deter the public from entering environmentally sensitive areas or to protect for safety concerns. Detention facilities located near roadways shall have guardrails or other safety measures acceptable to the governing agency.

10.4 INCREASED RUNOFF DETENTION BASIN DESIGN CRITERIA

Increased runoff mitigation may be required for new development and significant redevelopment that would increase peak flowrates on downstream properties where an engineered drainage system does not exist continually downstream of the site or may not have adequate capacity for the increase in runoff. Based on the location of the proposed development, the following criteria shall be used to mitigate increased runoff. Where increased runoff mitigation basins are designed as conjunctive/multi-use detention facilities, the Design Engineer is directed to pollutant control and hydromodification performance standards in the Water Quality Management Plan (WQMP) for the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable. Compliance with each performance criteria shall be determined and documented independently.

The entire area of proposed development will be routed through an increased runoff (detention) basin to mitigate increased runoff. All basins must have positive drainage, dead storage basins shall not be acceptable.

The methods described in the Riverside County Hydrology Manual must be followed to develop runoff hydrographs. Storms to be studied will include the 1-hour, 3-hour, 6-hour, and 24-hour duration events for the 2-year, 5-year, and 10-year return frequencies. Increased runoff basin(s) and outlet(s) sizing will ensure that none of these storm events have a higher peak discharge in the "after" condition than in the "before" condition.

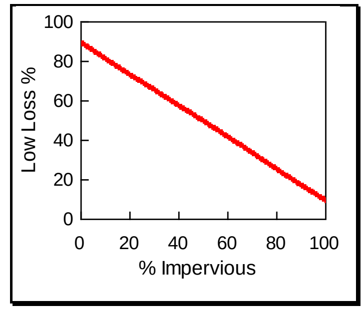
For the 2-year and 5-year events, the loss rate will be determined using an AMC I condition. For the 10-year event, AMC II will be used. Constant loss rates shall be used for the 1-hour, 3-hour, and 6-hour events. A variable loss rate shall be used for the 24-hour events.

Low Loss Rates will be determined using the following.

-Undeveloped Condition → *Low Loss = 90%*

-Developed Condition→ *Low Loss = 0.9 – (0.8 x % impervious)*

-Basin Site → *Low Loss = 10%*



When a detention basin is being used to mitigate increased runoff, where feasible, the onsite flows should be mitigated before combining with offsite flows to minimize the size of the detention facility required. If it is necessary to combine offsite and onsite flows into a detention facility, two separate conditions should be evaluated for each duration/return period for the before and after development combinations studied. The first condition is for the total tributary area (offsite plus onsite), and the second condition is for the area to be developed (onsite only). It must be clearly demonstrated that there is no increase in peak flow rates under either condition (tributary offsite and onsite area, or onsite area only) for each of the return period/duration combinations required to be evaluated. A single plot showing the pre-developed, post-developed, and routed hydrographs for both conditions for each storm considered shall be included with the submittal of the hydrology study.

No basin outlet pipe(s) may be less than 18 inches in diameter. Where a smaller conveyance is needed to control discharge, an orifice plate may be used to restrict outflow rates. Appropriate trash racks shall be provided for all outlets less than 48 inches in diameter and may also be required for larger outlets.

The basin(s) and outlet structure(s) must be capable of passing the 100-year storm without damage to the facility.

Increased runoff mitigation basins should be designed for conjunctive/multi-use and be incorporated into open space or park areas. Side slopes should be no steeper than 4:1 and depths should be minimized where public access is unrestricted.

A viable maintenance mechanism, acceptable to both the County and the District, must be provided for increased runoff detention facilities, including those being used for increased runoff mitigation. Generally, this would mean a CSA, landscape district, parks agency, or commercial property owners association. Residential homeowners associations would generally not be acceptable.

10.5 CONJUNCTIVE/MULTI -USE BASIN FACILITY

10.5.1 <u>Overview</u>

Conjunctive use or multi use means the use of a facility for two or more purposes. Because dry-pond detention facilities do not store water between storm events, it may be possible to have conjunctive uses for detention facilities that include , stormwater capture and recharge and active (includes sports activities) or passive (includes park areas) recreation. Conjunctive use of detention facilities is acceptable and encouraged when it is desirable and feasible.

This criterion addresses basins, or other storage facilities that are designed to meet multiple objectives including storm water management, increased runoff, and flood control objectives. Storm water management objectives include pollutant, and/or hydromodification controls. Increased runoff objectives include mitigating for post project increases in storm water runoff. Flood control objectives include the detention of the 100-year storm event to a protection level based on site specific conditions, such as mitigating a 100-year post project peak flow rate to the capacity of an existing downstream facility, or to match the 100-year pre-project flow rate (100-year peak flow attenuation/route-down to lower release rate). Note that when an above ground detention facility is used for both pollutant control and flood control, the water quality retention volume (e.g., volume for infiltration, which is the volume below lowest surface outlet) shall be provided in addition to the volume designated for flood storage.

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.

Typical allowable combinations of conjunctive uses are shown in the table below. Analyses should be conducted and documented independently for each basin type presented in the table below.

Table 10.1: Allowable Combinations of Conjunctive Use for Each Basin Type

Conjunctive Use	Basin Type				
	Water Quality Basin	Increased Runoff Basin	Hydromodification Basin	Local (Onsite) Detention Basin	Regional Detention Basin
Pollutant Control	X	X	X	X	
Increased Runoff		X	/	X	
Hydromodification		/	X	/	
Passive Recreation	X	X	X	X	/
Active Recreation				X	/
Flood Detention		/	/	X	X
Debris Catchment					/

X = Suitable, / = potentially suitable, blank = not suitable

For any proposed conjunctive use within regional detention basins proposed for District maintenance, the reviewer/designer shall obtain prior concurrence from the District before planning or designing for such joint uses. Where District maintenance is required, joint uses shall be subordinate to and not interfere with the District's operation and maintenance activity of the basin for flood control purposes. Note that the District will only maintain detention facilities with a regional benefit that route down the peak 100-year flow. Where there will be conjunctive use of a District maintained regional facility, additional maintenance entities will be required to maintain the conjunctive use features of the basin.

Where conjunctive/multi-use basins include pollutant and/or hydromodification features, the design of these facilities must meet the design criteria presented in this manual and the Design Handbook for Low Impact Development BMP for the Santa Ana, Santa Margarita, and Whitewater Regions of Riverside County, as applicable.

While design criteria for pollutant control, hydromodification, and increased runoff are not the focus of this manual, the design process for a conjunctive-use basin that combines one or more of these elements with flood detention purposes requires coordination between the storm water management engineer and the flood control engineer. The purpose of the information in this

Section is to clarify when the required storage volumes for the various objectives can overlap versus when they shall be additive, and to provide guidance for detention routing analysis for flood control.

10.5.2 Conjunctive/Multi-Use: Pollutant Control, Increased Runoff and Hydromodification

10.5.2.1 Conjunctive/Multi-Use Basin Storage Volume Requirements

Figure 10.9 and Figure 10.10 illustrate basin storage volume requirements for conjunctive/multi-use facilities for pollutant control, hydromodification, increased runoff mitigation and flood control. The following guidelines drive the basin volume requirements for the design of the conjunctive/multi-use detention facilities:

- The storage volume for flood control detention shall be provided in addition to the storage volume provided for pollutant control. This means that the flood control storage volume is the portion of the storage volume measured from the lowest above ground outlet of the basin and does not include the storage volume below the lowest above ground outlet (Figure 10.9 and Figure 10.10). This is to ensure that the flood control storage volume is independent from the infiltration volume of the pollutant control basin. Any portion of the storage volume infiltrated is not included as required volume for flood control storage.
- Where feasible, the storage volumes for pollutant control, hydromodification and/or increased runoff mitigation may overlap for the portion of the storage volume above ground (Figure 10.10).
- The storage volume for flood control detention is measured from the lowest above ground outlet (i.e., outlet that does not require water to pass through soil media to reach the outlet) (Figure 10.9 and Figure 10.10). Exception: if a portion of the storage volume measured from the lowest above ground outlet to another above ground outlet takes more than 72 hours to drain, then that portion of the volume may not be counted as storage volume for flood control, and the flood control storage volume must be measured from the first above ground outlet for which the volume above the outlet drains in less than 72 hours.
- Where feasible, the storage volume for flood control may overlap with the storage volume provided for increased runoff mitigation (Figure 10.10) and/or hydromodification control (Figure 10.10) Exception: If a portion of the storage volume for increased runoff mitigation and/or hydromodification control takes more than 72 hours to drain, then that portion of the increased runoff or hydromodification control storage volume may not be included as storage volume for flood control.

In Figure 10.9 and Figure 10.10, blue text accompanied by 'FC' indicates features related to flood control storage, purple text accompanied by 'HC' indicates features related to hydromodification control storage, red text accompanied by 'IR' indicates features related to increased runoff mitigation, and green text accompanied by 'PC' indicates items related to pollutant control storage.

Figure 10.9 Conjunctive/Multi-Use Facility for Pollutant Control and Flood Control Detention shows a facility for pollutant control and flood control detention without hydromodification control or increased runoff mitigation. Figure 10.9 illustrates the flood control storage volume being above the lowest above ground outlet (i.e., outlet that does not require water to pass through soil media to reach the outlet).

Figure 10.9: Conjunctive/Multi-Use Facility for Storm Water Management (Pollutant Control) and Flood Control Detention

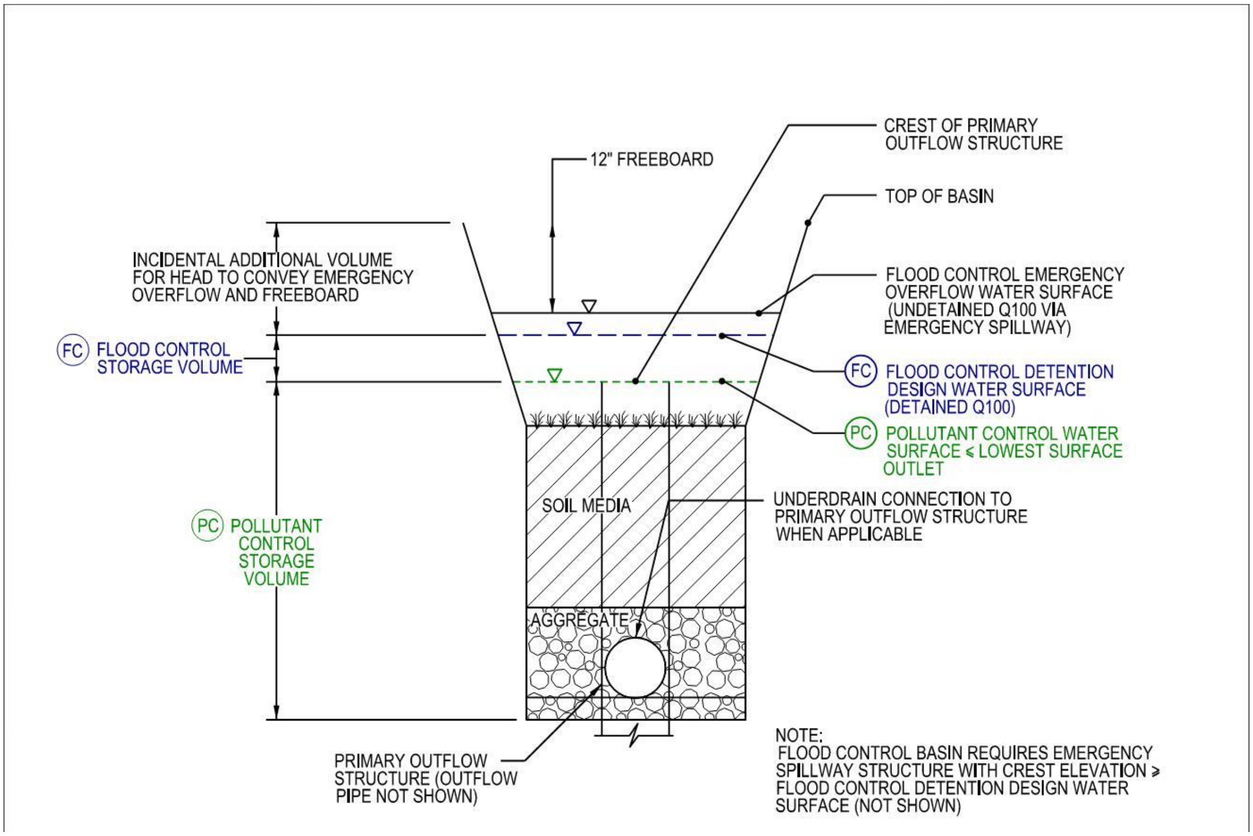
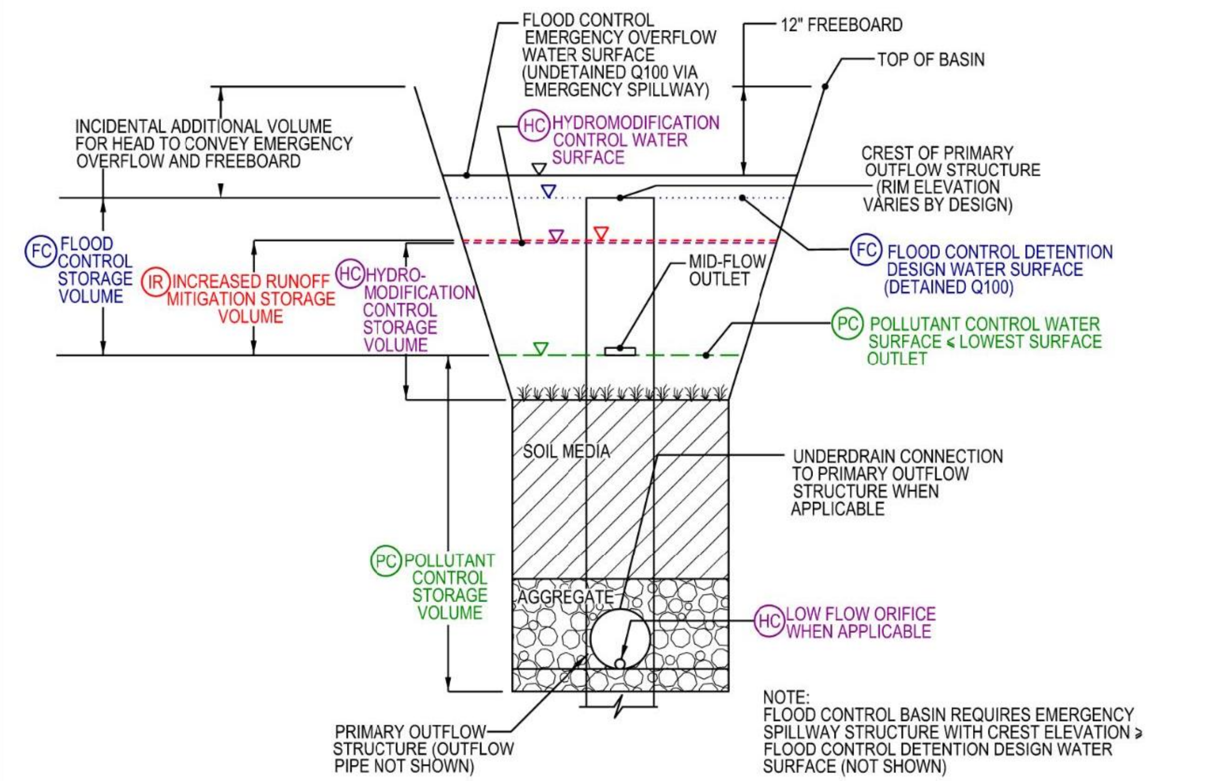


Figure 10.10 Conjunctive/Multi-Use Facility for Pollutant Control, Hydromodification Control, Increased Runoff Mitigation, and Flood Control Detention with Mid-Flow Outlet shows a facility for pollutant control, hydromodification control and/or increased runoff mitigation and flood control detention with a complex primary outflow structure. The flood control storage volume overlaps the portion of hydromodification control and/or increased runoff mitigation storage that is above the lowest above-ground outlet (i.e., outlet that does not require water to pass through soil media to reach the outlet).

Figure 10.10: Conjunctive/Multi-Use Facility for Pollutant Control, Hydromodification Control, Increased Runoff Migitation, and Flood Control Detention with Mid-Flow Outlet



CONJUNCTIVE USE FACILITY FOR STORM WATER MANAGEMENT AND FLOOD CONTROL: POLLUTANT CONTROL, HYDROMODIFICATION CONTROL AND FLOOD CONTROL DETENTION; WITH MID-FLOW OUTLET

10.5.2.2 Detention Routing Analysis for Flood Control Element of a Conjunctive Use Basin

To prepare the storage routing analysis for flood control, the engineer may either: (1) include stage-storage data for the entire depth of the facility from bottom to top but set the starting water surface for the analysis to the elevation of the lowest above-ground outlet, or (2) set the elevation of the lowest above-ground outlet as the datum for the "bottom" of the basin and exclude stage-storage data for elevations below the lowest above ground outlet. Stage-discharge data must only include outflow through the lowest above-ground outlet and any other outlets above the lowest above-ground outlet. Infiltration and outflow through the underdrain must be excluded from the flood control analysis.

10.5.3 Multi-Use: Parks

Recreational uses, such as a park, may be proposed within District-maintained basins. The primary function of a basin is for flood control use, therefore, there is a need for storage and expectation of flooding. See Section 1.4.2 for additional information.

10.5.4 Safety

On multi-use basins, all the safety topics discussed in Section 10.3.11 still apply, but wherever a basin is going to have passive or active public use, there are significant additional public safety considerations that must be addressed. Such considerations are beyond the scope of this manual, but should include at minimum: flood warning signage, gates to limit access during periods of flooding, special designs for inlet and outlet structures considering the likelihood that they are used as a 'playground', fencing at walls and steep slopes, etc.

10.6 REFERENCES

10.6.1 Internet Resources

California Department of Fish and Game, *https://www.wildlife.ca.gov*

California Department of Water Resources, Division of Safety of Dams (DSOD), *https://water.ca.gov/Programs/All-Programs/Division-of-Safety-of-Dams*

California State Water Resource Control Board, *http://www.waterboards.ca.gov*

U.S. Army Corps of Engineers, *https://www.usace.army.mil/*

U.S. Army Corps of Engineers Hydrologic Engineering Center,

https://www.hec.usace.army.mil/

U.S. Fish and Wildlife Service, *https://www.fws.gov*

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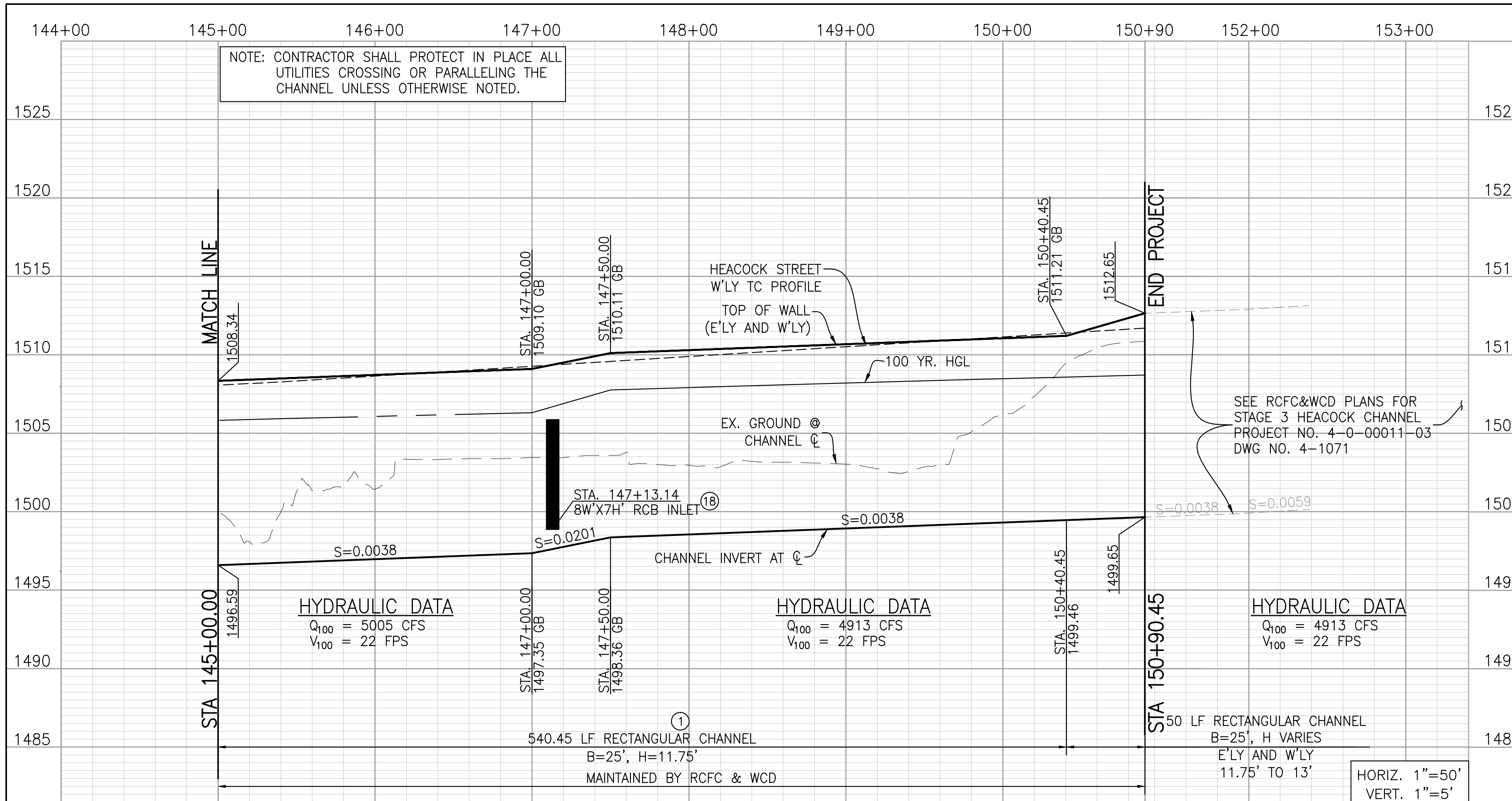
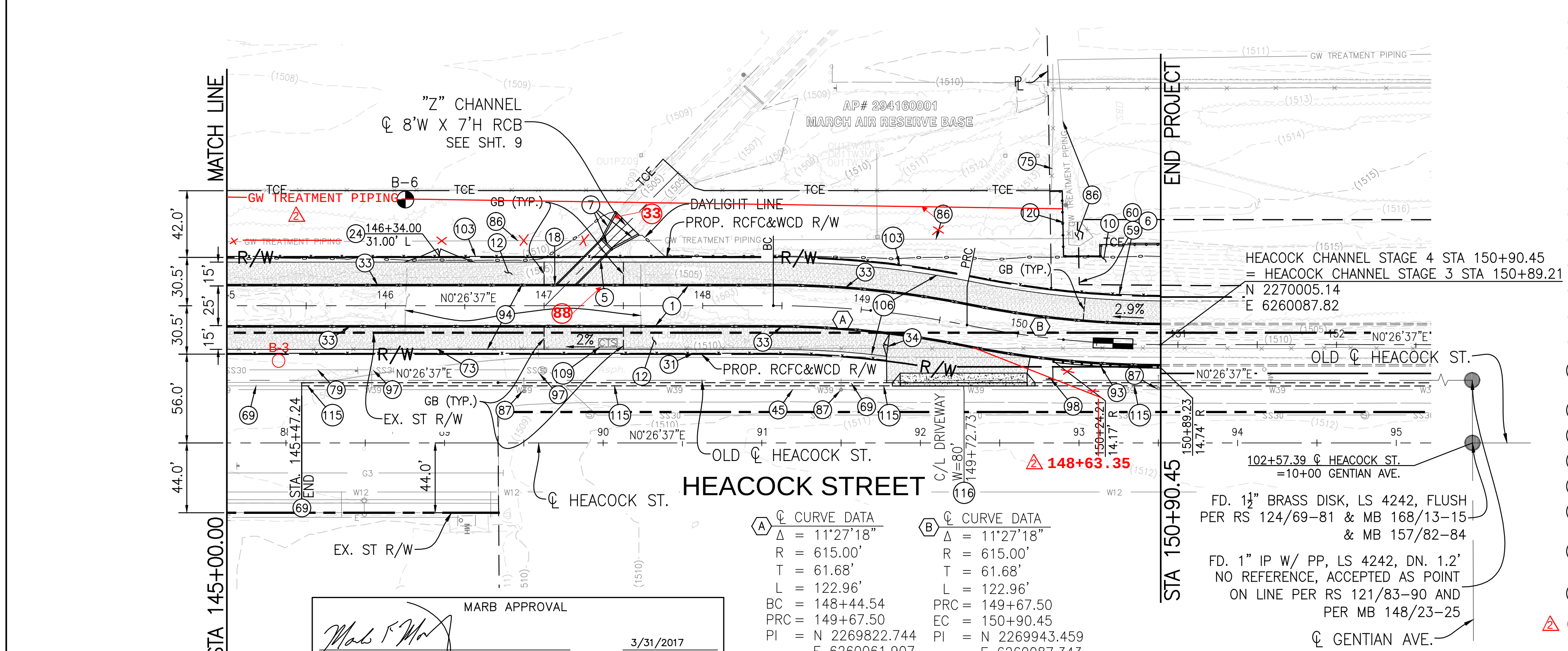
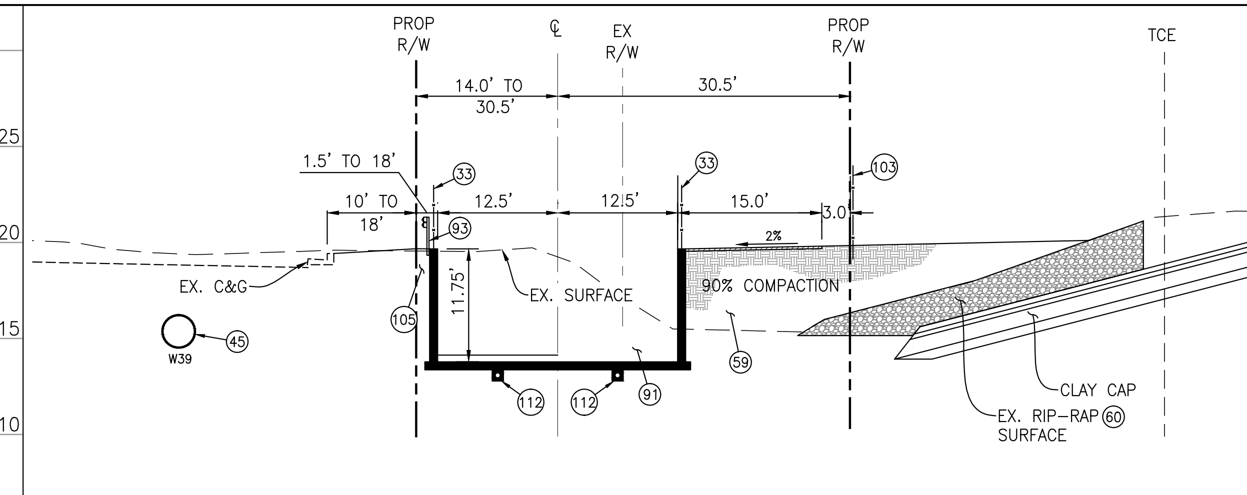
10.7 APPENDIX A







SECTION AT STA 150+60

TYP. FROM STA. 147+00 TO 150+90.45
SEE SHT. 12 FOR CHANNEL DETAILS

CONSTRUCTION NOTES:

1. CONSTRUCT 25' WIDE RECTANGULAR CONCRETE CHANNEL PER STD. CH 327 AND DETAILS ON SHEET 12. CHANNEL HEIGHT PER PROFILE.
5. CONSTRUCT 8' WIDE BY 7' HIGH RCB LATERAL CALTRANS STD. DWG. NO. D80 AND PARAPET WALL PER CALTRANS STD DWG NO D82
6. ON THE WEST SIDE OF THE CHANNEL, FROM STA. 150+30 TO STA. 150+90.45, PROTECT IN PLACE RIPRAP AND CLAY CAP ON LANDFILL PER RECORD DRAWINGS.
7. CONSTRUCT BOX CULVERT HEADWALL AND WARPED WINGWALL PER CALTRANS STD. D86A AND DETAIL B ON SHT. 11
10. FROM STA. 150+30 TO STA. 150+90.45, AS DIRECTED BY ENGINEER, REMOVE AND DISPOSE OF EXISTING VEGETATION ON THE CHANNEL BOTTOM AND WEST SIDESLOPE WITH CARE TO MINIMIZE DISRUPTION TO THE RIPRAP LAYER, CLAY CAP ATOP THE LANDFILL.
12. CONSTRUCT 3" THICK CRUSHED ROCK (1") ACCESS ROAD
18. CONSTRUCT CHANNEL WALL CONNECTION PER DETAIL B ON SHT. 11
24. INSTALL 24' WIDE DOUBLE SWING GATE (SWING AWAY FROM CHANNEL) PER UFC 4-022-03 SECURITY FENCES AND GATES SPECIFICATIONS AND DRAWING NO. UFC-700
31. INSTALL CHAINLINK FENCE PER STD. M801
33. INSTALL 3' CABLE RAILING PER CALTRANS STD. PLAN. B11-47
34. INSTALL 14' DOUBLE DRIVE GATE PER STD. M801
45. PROTECT IN PLACE EXISTING 39" WATER LINE
59. ON THE WEST SIDE OF THE CHANNEL, FROM STA. 150+30 TO STA. 150+90.45, BACKFILL WITH 90% RELATIVE COMPACTED SOIL.
60. ON THE WEST SIDE OF THE CHANNEL, FROM STA. 150+30 TO STA. 150+90.45, POUR CONCRETE SLURRY (1 SACK) TO BE FLUSH WITH THE TOP OF THE RIPRAP WITHIN LIMITS OF BACKFILL.
69. PROTECT IN PLACE EXISTING CURB & GUTTER
73. REMOVE AND DISPOSE OF EXISTING FENCE
75. PROTECT IN PLACE EXISTING FENCE
79. PROTECT IN PLACE EXISTING 30" SEWER
86. PROTECT IN PLACE EXISTING GROUNDWATER TREATMENT PIPING
87. PROTECT IN PLACE EXISTING STREET LIGHT
91. EXCAVATE RECTANGULAR CHANNEL PER PAYLINE DETAILS ON SHEET 12
93. INSTALL METAL BEAM GUARD RAILING AND STEEL POSTS PER CALTRANS STD. PLAN RSP A77A2
94. REMOVE AND DISPOSE OF EXISTING CONCRETE JUNCTION STRUCTURE (150 FT.)
97. PROTECT EXISTING MANHOLE IN PLACE
98. INSTALL "AUTHORIZED VEHICLES ONLY" (R5-11) SIGN PER CA MUTCD
103. INSTALL 6' HIGH (MATCH EXISTING) AIRFORCE SECURITY FENCE WITH SINGLE 45' BARBED WIRE ARM (BARBS TOWARD THE CHANNEL) PER UFC 4-022-03 SECURITY FENCE AND GATES SPECIFICATIONS AND DRAWING NO. UFC-702
105. BACKFILL PER LIMITS SHOWN ON PLAN, PROFILE AND DETAILS ON SHEET 12
106. REMOVE AND DISPOSE OF EXISTING SHRUBS, VEGETATION AND ROCKS ALONG CHANNEL BOTTOM AND SIDE SLOPES
109. EXISTING CTS UTILITY TO BE RELOCATED BY EMWD.
112. CONSTRUCT SUBDRAIN SYSTEM PER STD CH332. NUMBER AND LOCATION PER SECTION HEREON
115. FROM APPROXIMATELY STATION 145+50 TO STA 150+90.45, CONSTRUCT PROPOSED STREET IMPROVEMENTS PER HEACOCK CHANNEL STAGE 3, DWG. NO. 4-1071, SHEET 23
116. CONSTRUCT DRIVEWAY APPROACH PER CITY OF MORENO VALLEY STD. DWG. NO. MVSI-111A-0 AND MODIFY ENTRY WIDTH OF 80'. USE MINIMUM DISTANCES FOR A 10' RIGHT OF WAY
120. PLACE TEMPORARY FENCING ALONG TCE WITHIN LANDFILL AREA AS DIRECTED BY ENGINEER

⚠ 88. **MANHOLE STEPS AND GATE.**

RECORD DRAWINGS
APPROVED:
DATE: 9/19/17

MARB APPROVAL
MARK MORITZ – 452 AMW CHIEF OF ENGINEERING
DATE: 3/31/2017

BENCH MARK
B.M. NO. Z-9905
FD 1 1/4" IP. W/PLUG STMPD. "H-G SURVEY PLS 5137" DN. 0.44'. NEC CACTUS AVE/HEACOCK ST
1548.67 EL

Don't Dig...Until You Call:
U.S.A. Toll Free: 1-800-227-2600 for the location of buried utility lines. Don't disrupt vital services.
TWO WORKING DAYS BEFORE YOU DIG

ALBERT A. WEBB ASSOCIATES
ENGINEERING CONSULTANTS
3788 McCRAY STREET
RIVERSIDE CA. 92506
PH. (951) 686-1070
FAX (951) 788-1256
DATE: 3/31/2017
DESIGNED BY: RFS | DRAWN BY: RFS | CHECKED BY: JCC

REGISTERED PROFESSIONAL ENGINEER JOSEPH C. CALDWELL NO. C67239 CIVIL STATE OF CALIFORNIA

REF.	REVISIONS DESCRIPTION	APPR.	DATE
⚠	NOTE REVISION, REVISE GW TREATMENT PIPING LOCATION		9/19/17

RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT P8\213409
RECOMMENDED FOR APPROVAL BY: DATE: 3/31/2017
APPROVED BY: DATE: 3/31/2017

HEACOCK CHANNEL
(SUNNYMEAD MDP LINE B)
STAGE 4
STA 145+00.00 TO STA 150+90.45

PROJECT NO. 4-0-00011-04
DRAWING NO. 4-1088
SHEET NO. 6 OF 12

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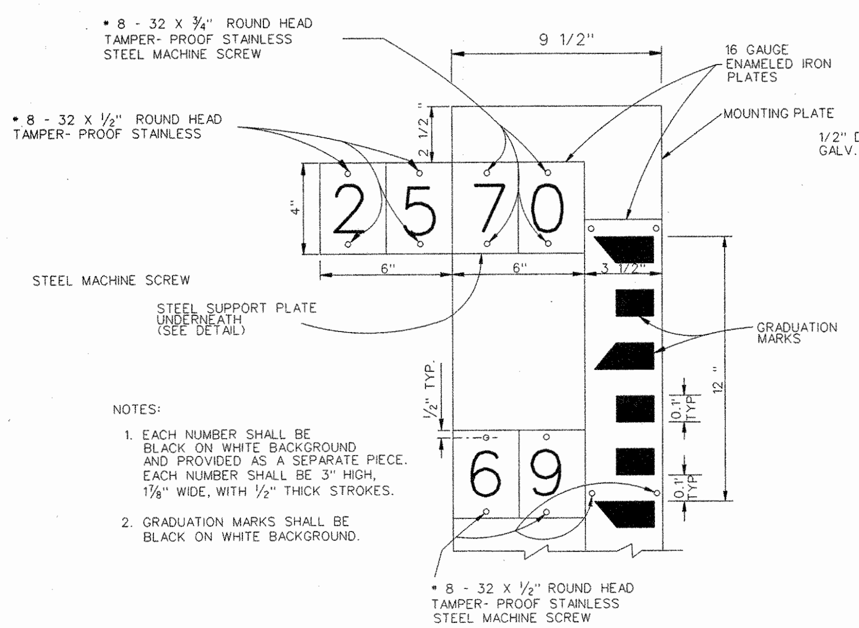
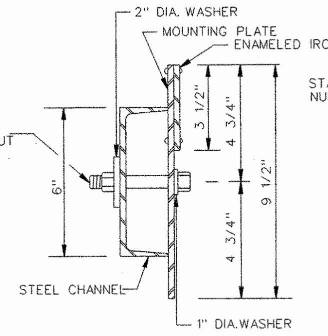


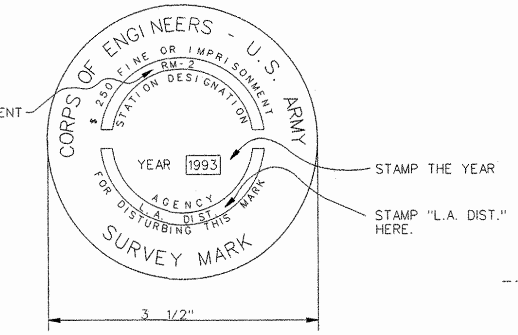
NOTES:

1. EACH NUMBER SHALL BE BLACK ON WHITE BACKGROUND AND PROVIDED AS A SEPARATE PIECE. EACH NUMBER SHALL BE 3" HIGH, 1 7/8" WIDE, WITH 1/2" THICK STROKES.
2. GRADUATION MARKS SHALL BE BLACK ON WHITE BACKGROUND.

UPPER PORTION OF GAGE PLATE
(WITH ENAMELED PLATES ATTACHED)
NOT TO SCALE



SECTION B-B AT BOLTS
NOT TO SCALE

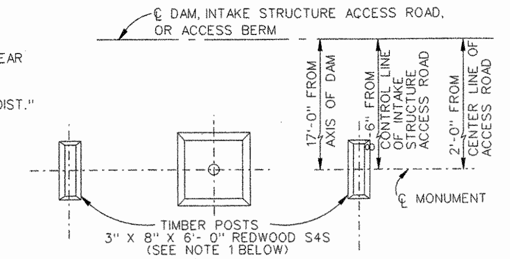


TOP VIEW OF BRONZE DISK

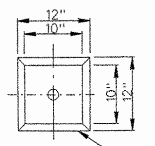
NOTES:

1. BRONZE DISKS TO BE FURNISHED BY CONTRACTING OFFICER.
2. SIZE OF LETTERS AND NUMBERS STAMPED ON DISK TO 3/16" IN HEIGHT.
3. METAL STAMPING OUTFITS SHALL BE USED FOR STAMPING.

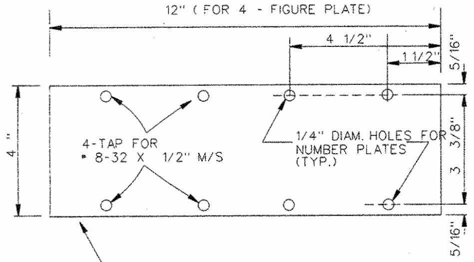
SURVEY MONUMENT
NOT TO SCALE



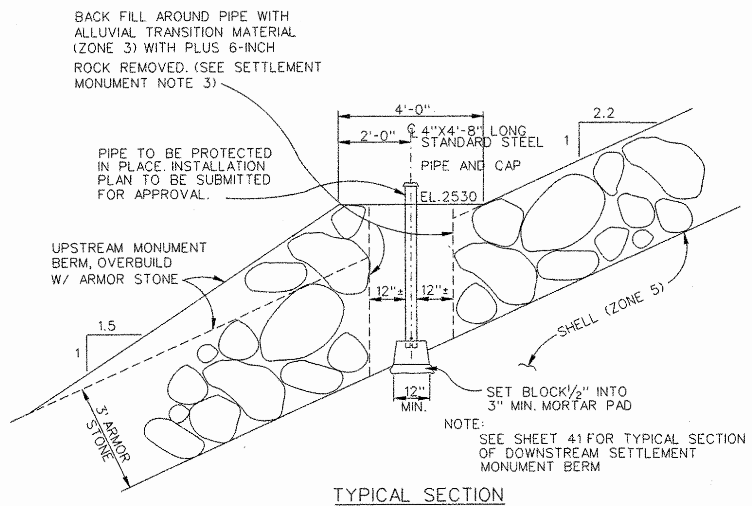
PLAN



PLAN

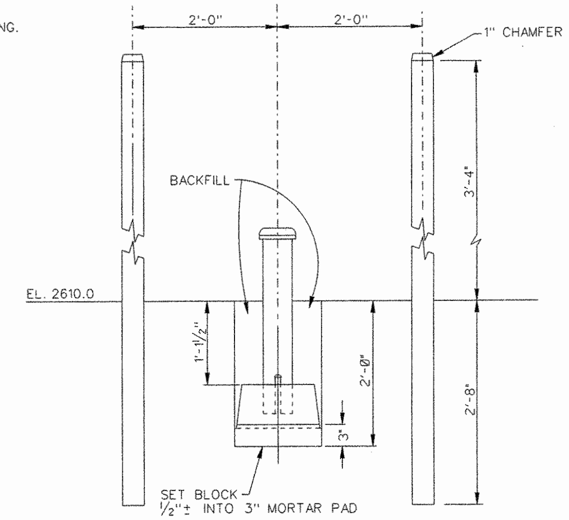


STEEL SUPPORT PLATE DETAIL
NOT TO SCALE

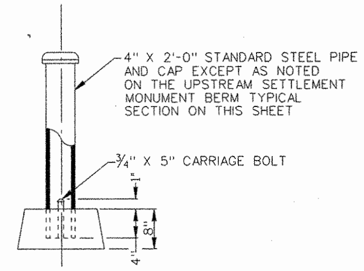


NOTE:
SEE SHEET 41 FOR TYPICAL SECTION OF DOWNSTREAM SETTLEMENT MONUMENT BERM

TYPICAL SECTION
UPSTREAM SETTLEMENT MONUMENT BERM
NOT TO SCALE



ELEVATION
MONUMENT INSTALLATION

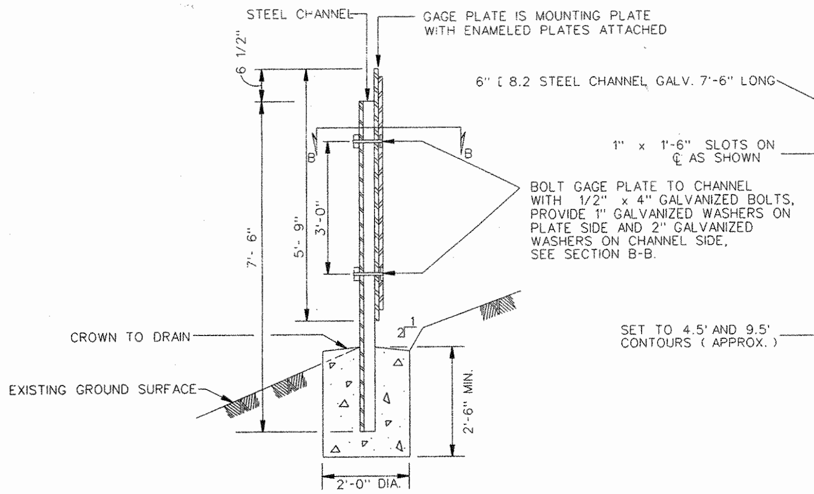


ELEVATION
MONUMENT DETAILS

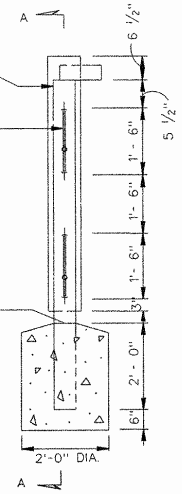
NOTES:

1. TIMBER POSTS SHALL BE REQUIRED FOR MONUMENTS INSTALLED ON THE CREST (EL. 2610) AND ALONG THE INTAKE STRUCTURE ACCESS ROAD.
2. THE DIMENSIONS SHOWN FOR MONUMENT INSTALLATION ARE APPLICABLE ON THE CREST, THE INTAKE STRUCTURE ROAD, AND THE DOWNSTREAM MONUMENT BERMS.
3. BACKFILL FOR MONUMENTS INSTALLED ON THE DOWNSTREAM ACCESS BERMS SHALL CONSISTS OF ROCKFILL MATERIAL (ZONE 4B) WITH PLUS 6- INCH ROCK REMOVED.

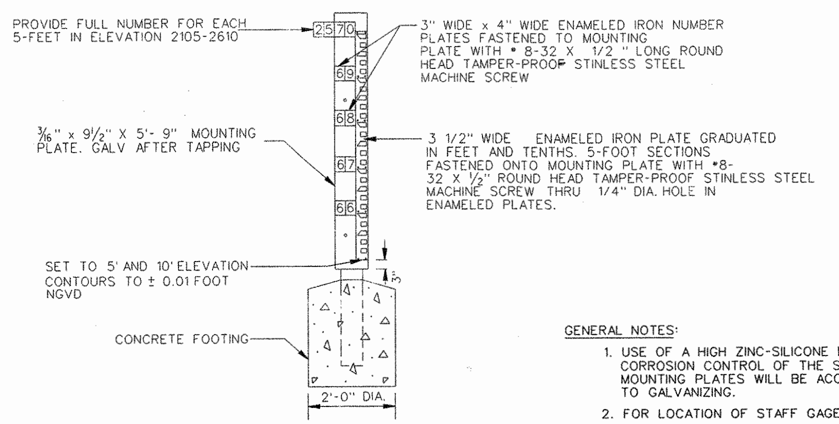
SETTLEMENT MONUMENT
NOT TO SCALE



SECTION A-A



REAR VIEW



FRONT VIEW

GENERAL NOTES:

1. USE OF A HIGH ZINC-SILICONE INORGANIC COATING FOR CORROSION CONTROL OF THE STEEL CHANNEL AND MOUNTING PLATES WILL BE ACCEPTABLE AS AN OPTION TO GALVANIZING.
2. FOR LOCATION OF STAFF GAGES, SEE SHEET 42.
3. FOR LOCATION OF SETTLEMENT MONUMENTS, SEE SHEET 36.

STAFF GAGE DETAILS
NOT TO SCALE

COMPUTER AIDED DESIGN & DRAFTING
STAFF.DGN

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

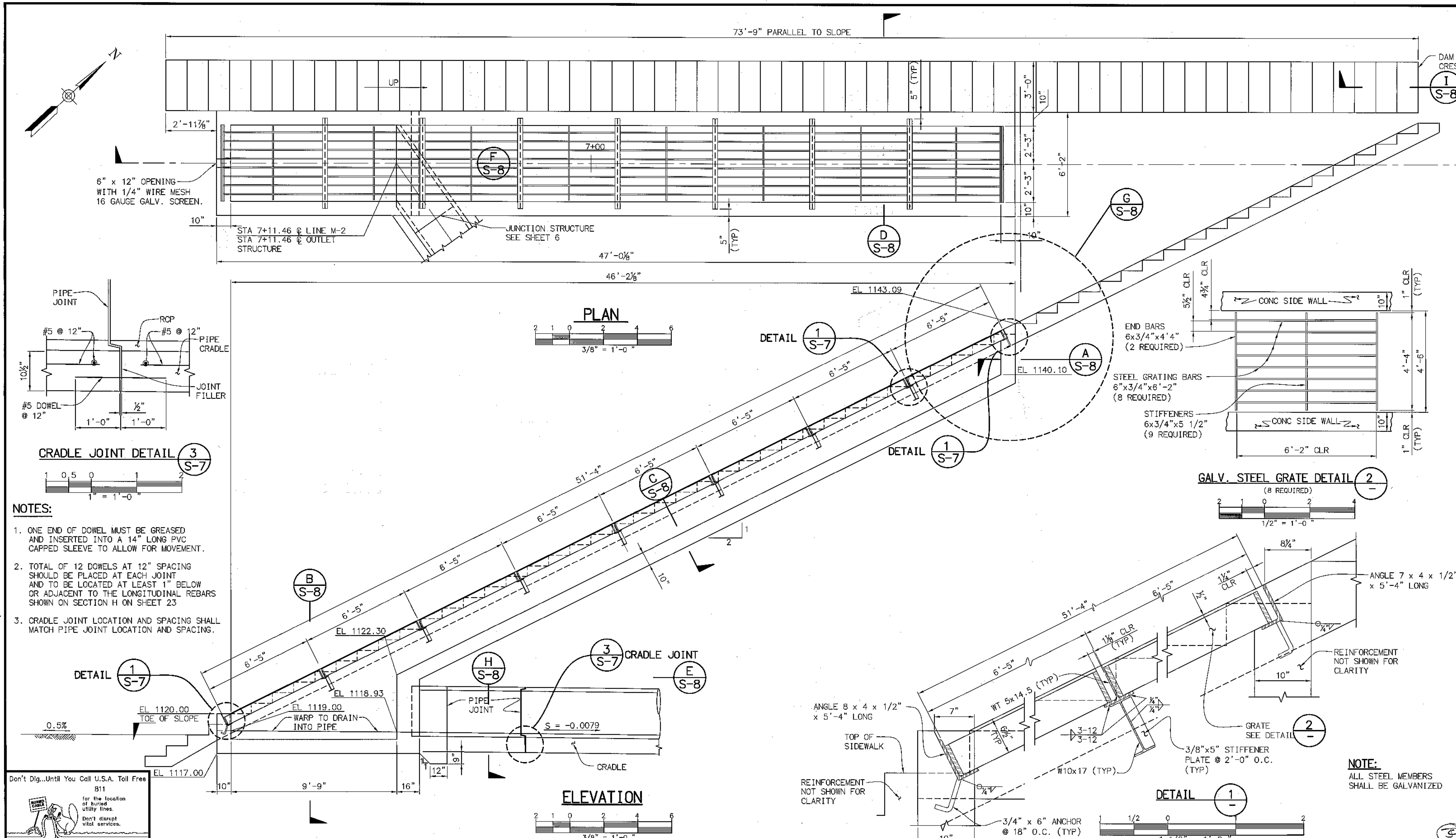
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM
SAN BERNARDINO COUNTY, CALIFORNIA

DESIGNED BY: R.K
DRAWN BY: A.C
CHECKED BY: MPC

SEVEN OAKS DAM
DAM AND APPURTENANCES
STAFF GAGE,
SETTLEMENT AND SURVEY MONUMENTS DETAILS

SUBMITTED BY: ROBERT L. HALL, P.E.
DATE APPROVED: 1 MAR. 1993
SPEC. NO. DACW09- 93B-0010
DISTRICT FILE NO. 243/252
SHEET 43 OF SHEETS



NOTES:

1. ONE END OF DOWEL MUST BE GREASED AND INSERTED INTO A 14" LONG PVC CAPPED SLEEVE TO ALLOW FOR MOVEMENT.
2. TOTAL OF 12 DOWELS AT 12" SPACING SHOULD BE PLACED AT EACH JOINT AND TO BE LOCATED AT LEAST 1" BELOW OR ADJACENT TO THE LONGITUDINAL REBARS SHOWN ON SECTION H ON SHEET 23
3. CRADLE JOINT LOCATION AND SPACING SHALL MATCH PIPE JOINT LOCATION AND SPACING.

Don't Dig...Until You Call U.S.A. Toll Free 811 for the location of buried utility lines. Don't disrupt vital services. TWO WORKING DAYS BEFORE YOU DIG

BENCH MARK: 2-1/2" BRASS DISK STAMPED "C-125" SET IN THE TOP OF CURB, LOCATED 5' SOUTH OF THE B.C.R. OF THE SOUTHEASTERLY CURB RETURN OF THE INTERSECTION OF BORDER AVENUE AND MABEY CANYON ROAD

ELEVATION=1083.414 (NGVD29)

REF.	REVISIONS DESCRIPTION	ENGINEER APPR.	DATE	RCFC&WCD APPR.	DATE

DESIGNED BY: F. SANCHEZ
DRAWN BY: J. SALDANA
DATE DRAWN: 05/19/14

APPROVED BY: RBF CONSULTING A Company 14725 ALTON PARKWAY IRVINE, CALIFORNIA 92618-2027 949.472.3505 • FAX 949.472.8373 • www.RBF.com
FABIO SANCHEZ 05/19/14 DATE:

REGISTERED PROFESSIONAL ENGINEER FABIO SANCHEZ No. 52764 Exp. 12/31/14 CIVIL STATE OF CALIFORNIA

RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
RECOMMENDED FOR APPROVAL BY: DATE: 6/3/2014
APPROVED BY: Mark H. Wills DATE: 7/10/2014

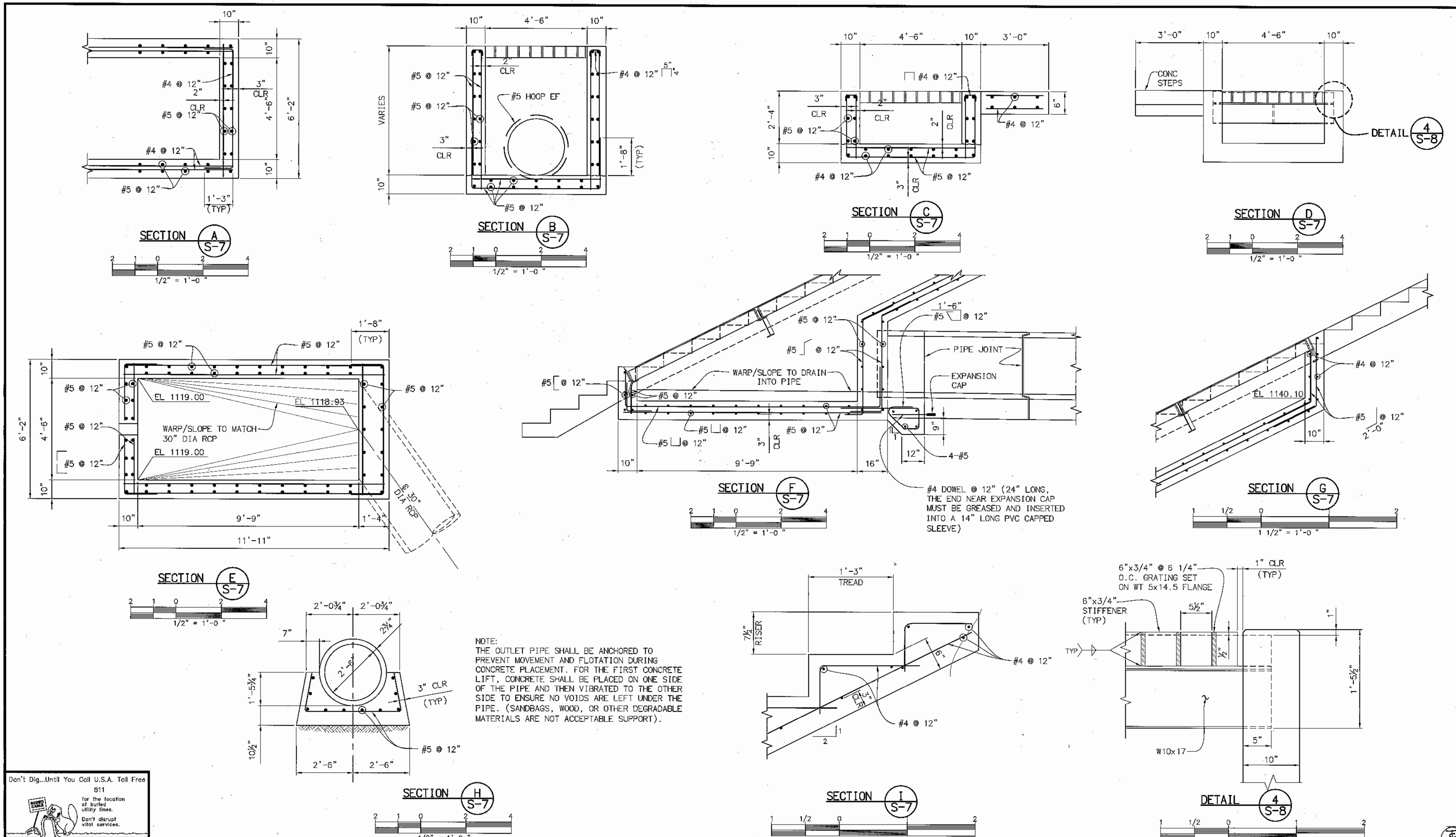
CITY OF CORONA ENGINEERING DEPARTMENT APPROVED BY:
CITY ENGINEER NELSON D. NELSON, P.E. DATE: 5-20-14
RECOMMENDED DATE:

MABEY CANYON DEBRIS BASIN STAGE 1

STRUCTURAL OUTLET STRUCTURE – SECTIONS 1

PROJECT NO. 2-0-00041-01
DRAWING NO. 2-0438
SHEET NO. 22 OF 23

H:\pdata\10104629.002\CADD\Struc\DIV\Mabey Canyon\4629-ST-S7_CO-01.dwg 05/19/14 - 1:28pm tkeith



NOTE:
THE OUTLET PIPE SHALL BE ANCHORED TO PREVENT MOVEMENT AND FLOTATION DURING CONCRETE PLACEMENT. FOR THE FIRST CONCRETE LIFT, CONCRETE SHALL BE PLACED ON ONE SIDE OF THE PIPE AND THEN VIBRATED TO THE OTHER SIDE TO ENSURE NO VOIDS ARE LEFT UNDER THE PIPE. (SANDBAGS, WOOD, OR OTHER DEGRADABLE MATERIALS ARE NOT ACCEPTABLE SUPPORT).

Don't Dig...Until You Call U.S.A. Toll Free 811 for the location of buried utility lines. Don't disrupt vital services. TWO WORKING DAYS BEFORE YOU DIG

BENCH MARK: 2-1/2" BRASS DISK STAMPED "C-125" SET IN THE TOP OF CURB, LOCATED 5' SOUTH OF THE B.C.R. OF THE SOUTHEASTERLY CURB RETURN OF THE INTERSECTION OF BORDER AVENUE AND MABEY CANYON ROAD
ELEVATION=1083.414 (NGVD29)

REF.	REVISIONS DESCRIPTION	ENGINEER APPR.	DATE	RCFC&WCD APPR.	DATE

DESIGNED BY: F. SANCHEZ
DRAWN BY: J. SALDANA
DATE DRAWN: 05/19/14

APPROVED BY: RBF CONSULTING A Baker Company
14725 ALTON PARKWAY IRVINE, CALIFORNIA 92618-2027
FABIO SANCHEZ 05/19/14 DATE:

REGISTERED PROFESSIONAL ENGINEER FABIO SANCHEZ No. 52764 Exp. 12/31/14 CIVIL STATE OF CALIFORNIA

RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
RECOMMENDED FOR APPROVAL BY: DATE: 6/3/2014
APPROVED BY: DATE: 7/10/2014

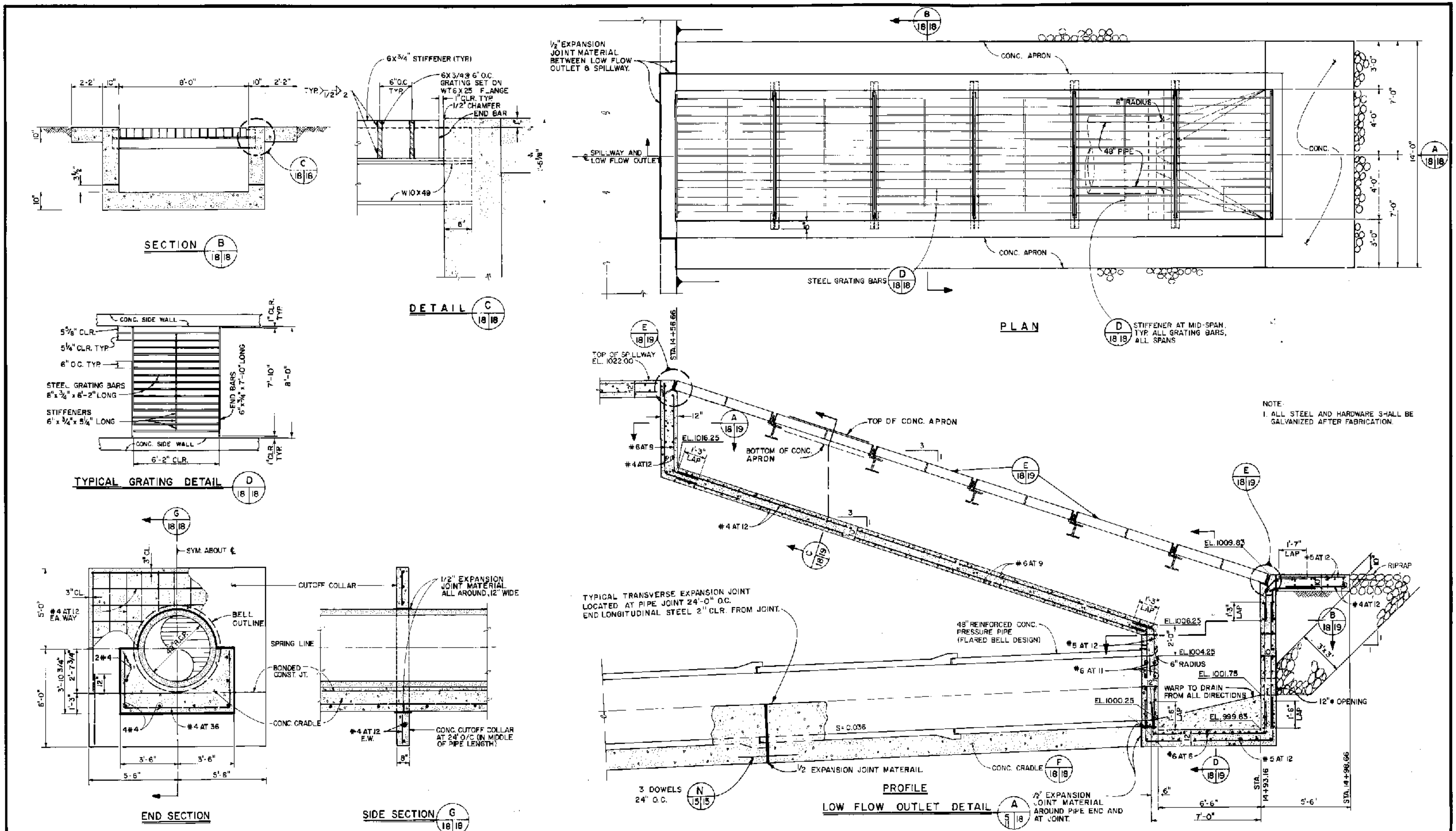
CITY OF CORONA ENGINEERING DEPARTMENT
APPROVED BY: 5-20-14
CITY ENGINEER NELSON D. NELSON, P.E. DATE:
RECOMMENDED DATE:

MABEY CANYON DEBRIS BASIN STAGE 1

STRUCTURAL
OUTLET STRUCTURE – SECTIONS 2

PROJECT NO. 2-0-00041-01
DRAWING NO. 2-0438
SHEET NO. 23 OF 23

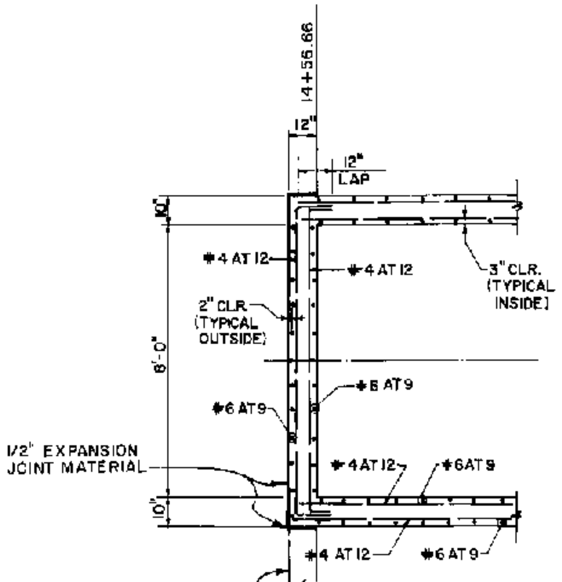
S-8



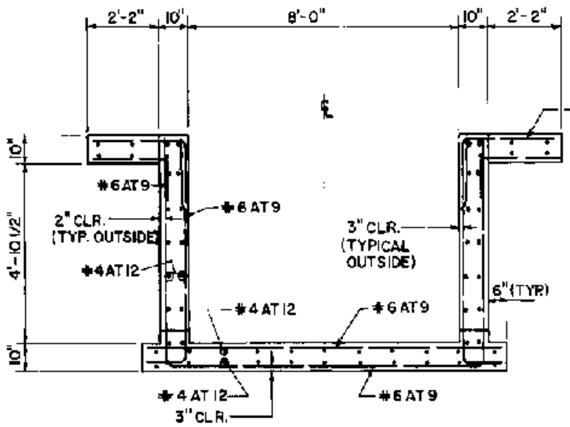
CONC. CRADLE & CUTOFF COLLAR DETAIL F 18|18

BENCH MARK	ALDERMAN SWIFT & LEWIS CONSULTING ENGINEERS — APPROVED BY; DATE 11-15-78; REGISTERED CIVIL ENGINEER NO. 11558	REVISIONS: DRAWING REVISED 11-27-78; DRAWING REVISED 12-7-78; No Change	RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT — DESIGNED BY; DRAWN BY; DATE DRAWN; CHECKED BY; APPROVED BY: CHIEF ENGINEER R.E. NO. 12400; DATE: 11-29-78	OAK STREET DEBRIS BASIN — LOW FLOW OUTLET DETAILS — PROJECT NO. 2-0-070; DRAWING NO. 2-162; SHEET NO. 18 OF 26
SCALE NO SCALE		REF. / DESCRIPTION / APPR. DATE		
DATE:				

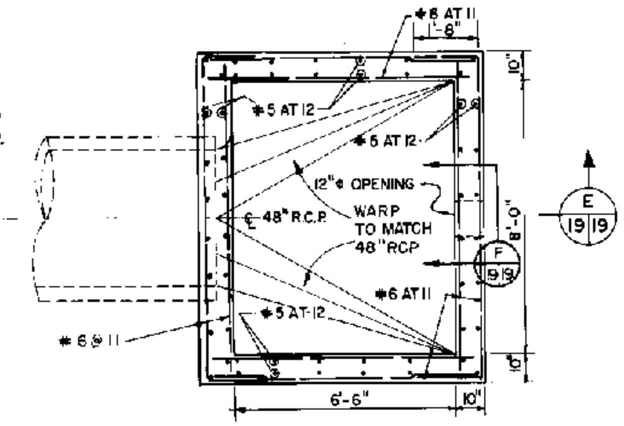
AS BUILT



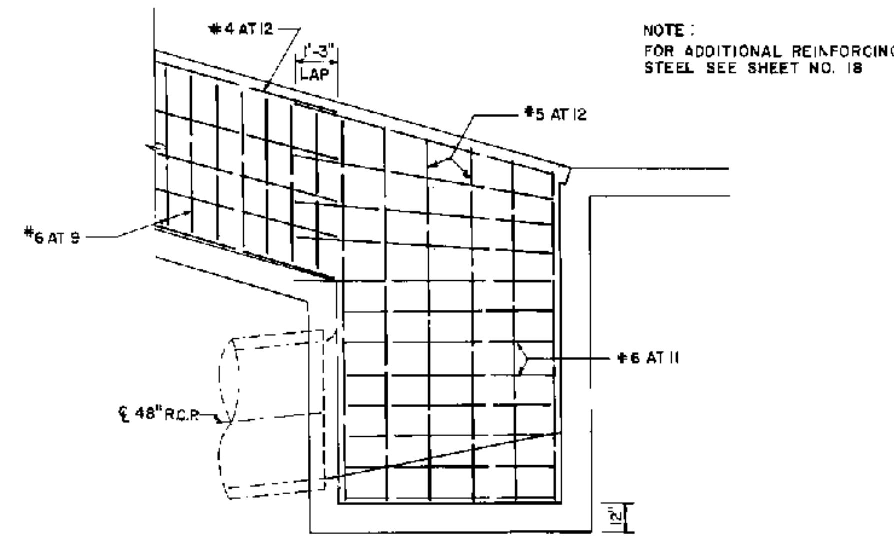
SECTION A 18|19



SECTION C 18|19

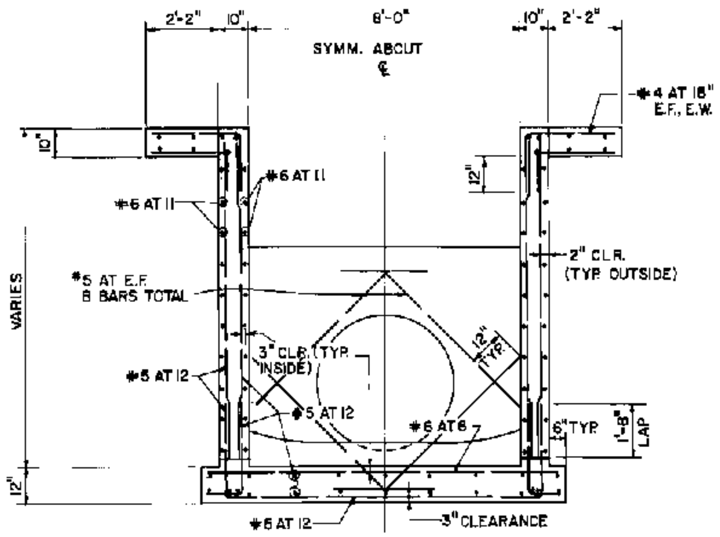


SECTION B 18|19

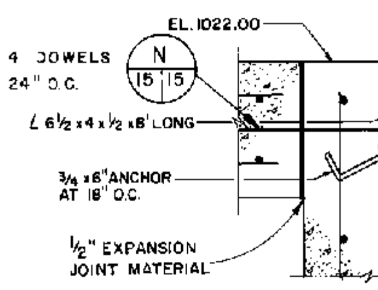
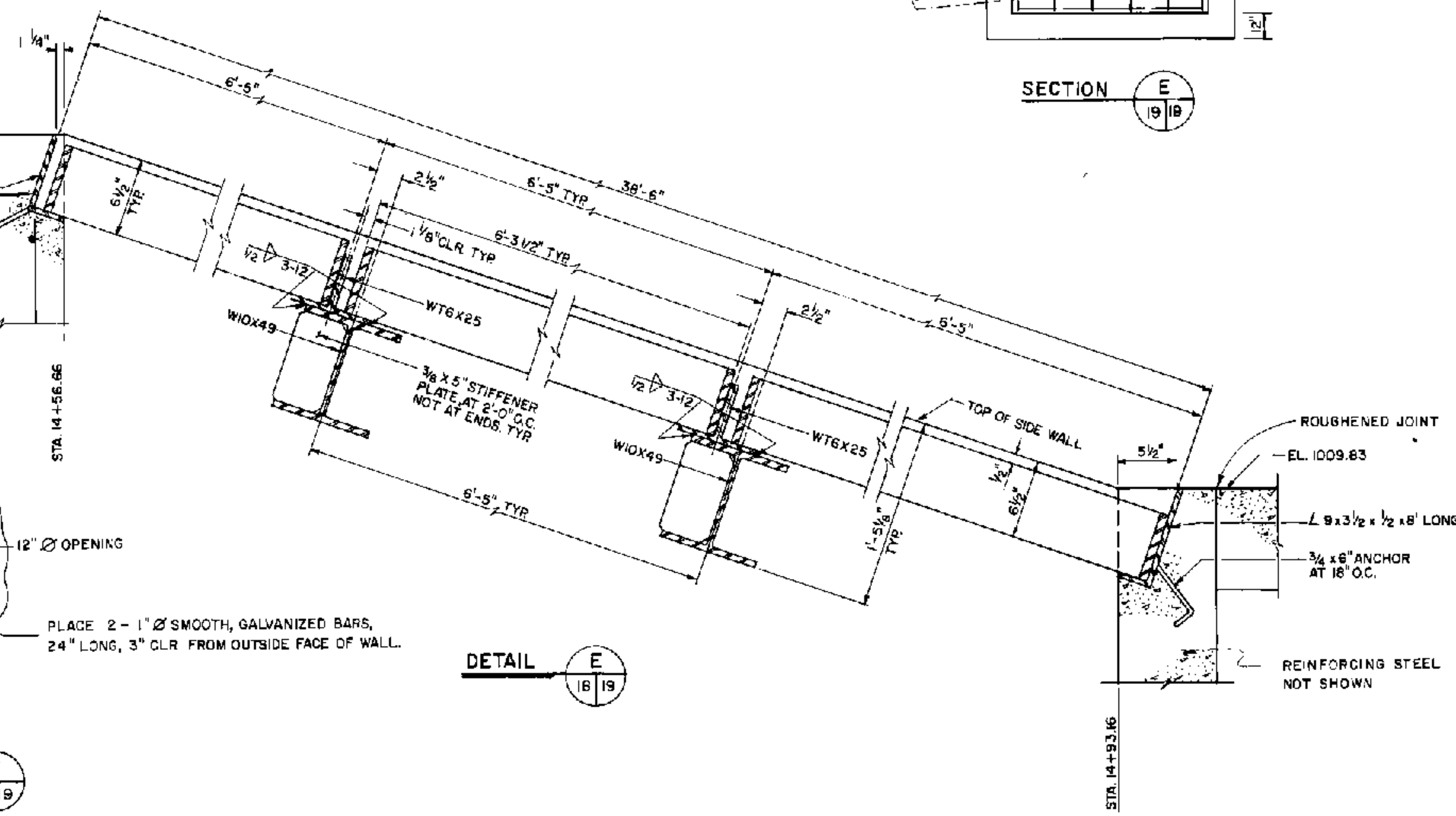


NOTE:
FOR ADDITIONAL REINFORCING STEEL SEE SHEET NO. 18

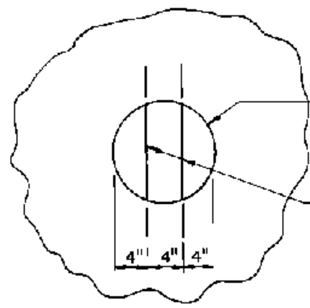
SECTION E 19|19



SECTION D 18|19

DETAIL E 18|19



12" OPENING DETAIL F 19|19

BENCH MARK		APPROVED BY	REVISIONS			RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT	OAK STREET DEBRIS BASIN	PROJECT NO. 2-0-070
SCALE: NO SCALE	ALDERMAN SWIFT & LEWIS CONSULTING ENGINEERS	DATE 11-15-78	DRAWING REVISED 11-27-78			DESIGNED BY: / DRAWN BY: / DATE DRAWN: / CHECKED BY:	LOW FLOW OUTLET STRUCTURAL DETAILS	DRAWING NO. 2-162
DATE:		REGISTERED CIVIL ENGINEER NO. 11643	DRAWING REVISED 12-11-78			APPROVED BY: CHIEF ENGINEER R.E. NO. 12400 DATE: 11-28-78		SHEET NO. 19 OF 26
			REF.	No Change / DESCRIPTION	APPR. / DATE			

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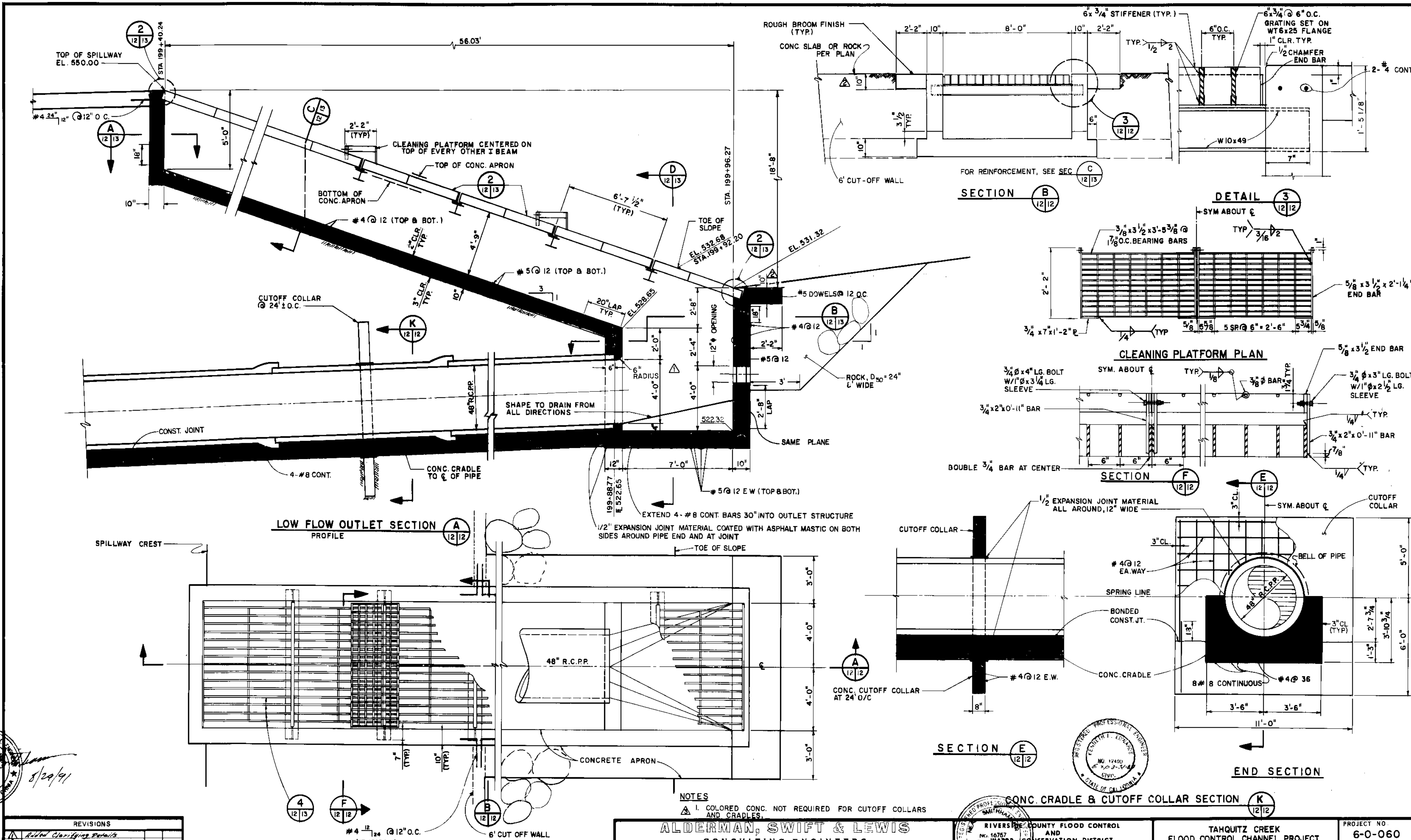

NOTES

1. COLORED CONC. NOT REQUIRED FOR CUTOFF COLLARS AND CRADLES.

AS BUILT

APPROVED BY: [signature] DATE 8/5/91

8/29/91

REVISIONS			
1	Added Clarifying Details		
2	Increased Concrete Thickness		
REF.	DESCRIPTION	APPR	DATE

ALDERMAN, SWIFT & LEWIS
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SOUTH PASADENA · SANTA ANA · PALM SPRINGS
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DESIGNED BY V.G. — DRAWN BY G.G. — CHECKED BY — DATE — SCALE NO SCALE — FILE NO. 577.7

RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT

DESIGNED BY: — DRAWN BY: — DATE DRAWN: — CHECKED BY:

APPROVED BY: Kenneth L. Edwards, CHIEF ENGINEER R.E. NO. 12400

DATE: 12-11-89

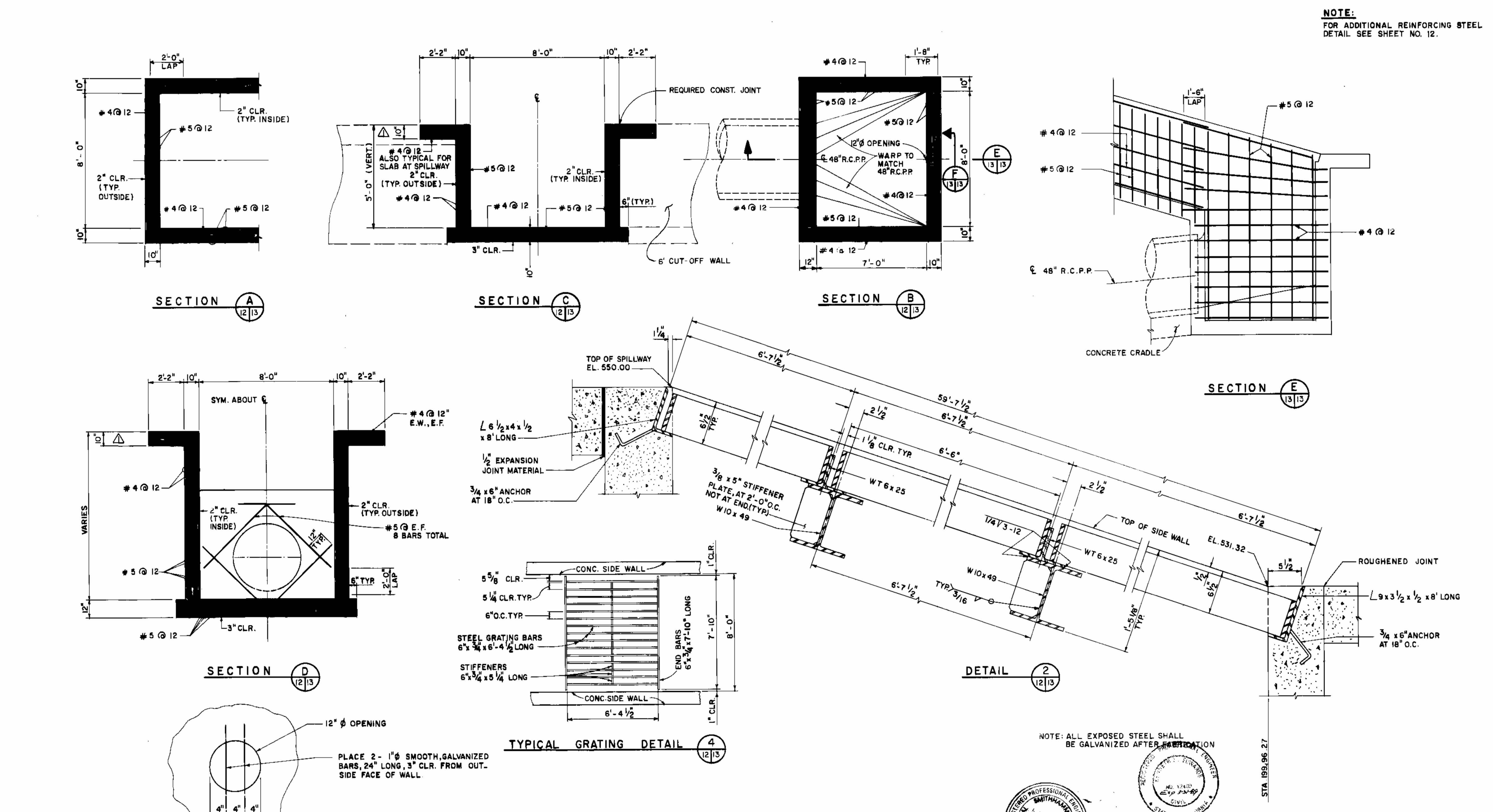
TAHQUITZ CREEK FLOOD CONTROL CHANNEL PROJECT STAGE II

LOW FLOW OUTLET STRUCTURAL DETAILS

PROJECT NO. 6-0-060

DRAWING NO. 6-251

SHEET NO. 12 OF 26



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DATE 8/15/91

REVISIONS		
△ Increased Concrete Thickness		
REF.	DESCRIPTION	APPR DATE

ALDERMAN, SWIFT & LEWIS
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RIVERSIDE COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT
APPROVED BY: Kenneth L. Edwards
CHIEF ENGINEER R.E. NO. 12400
DATE: 12-11-81

TAHQUITZ CREEK
FLOOD CONTROL CHANNEL PROJECT
STAGE II
LOW FLOW OUTLET
STRUCTURAL DETAILS

PROJECT NO. 6-0-060
DRAWING NO. 6-251
SHEET NO. 13 OF 26

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11 DEBRIS BARRIERS/BASINS

11.1 INTRODUCTION

The use of debris barriers (trash racks, bollards) and debris basins to protect channels, culverts, and underground storm drains should be carefully considered on a case-by-case basis.

Debris barriers are typically grates, bollards or other methods installed at the entrance to the channel or conduit to physically block debris from entering. Properly used and managed debris barriers are an important feature to prevent clogging and associated flooding. If improperly used or maintained, or fi the upstream watershed has the potential to deliver significant amounts of sediment or debris, the debris barrier may become clogged quickly, preventing water from entering the conduit before maintenance could occur to restore the capacity.

In cases where significant quantities of sediment or debris are anticipated, debris basins are typically required. The basins provide sufficient storage volume to trap the sediment and debris, helping prevent clogging of downstream debris barriers and conduits.

11.2 GENERAL DESIGN CRITERIA

The purpose of debris barriers and basins is to capture and trap sediment, rock, or other debris in a way that will 1) not plug the downstream culvert or storm drain, and 2) be configured for easy cleanout and removal.

Several types of structures might be used to reduce the effect of debris on culverts and pipes. The first step in selecting the appropriate debris control device is to categorize the type of debris inflow. The U.S. Department of Transportation (USDOT, 1971) has developed the following classification of debris:

- Light floating debris – small limbs or sticks, orchard prunings, tules, and refuse
- Medium floating debris – limbs or large sticks
- Large floating debris – logs or trees
- Flowing debris – heterogeneous fluid mass of clay, silt, sand, gravel, rock, refuse, or sticks
- Fine detritus – fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity
- Coarse detritus – coarse gravel or rock fragments carried as channel bedload at flood stage

Table 11.1 shows a matrix of commonly used structures and their generally acceptable use depending on expected debris characteristics. The following Sections provide a general discussion of some of these structures.

Table 11.1: Classification of Debris

Debris Classification	Type of Structure			
	Debris Rack	Debris Posts	Permanent Desiltation Basin	Debris Basin
Light Floating Debris	X			X
Medium Floating Debris	X	X		X
Heavy Floating Debris		X		X
Flowing Debris			X	X
Fine Detritus			X	X
Coarse Detritus			X	X
Boulders		X		X

Where there is potential for debris production in the upstream watershed and where there are bridge piers or other obstructions in the channel, the engineer should consider the reduction in hydraulic capacity caused by the accumulation of debris on the obstruction. In absence of other analytical methods, the engineer should assume an additional 1 to 2 feet of debris build up on each side of the obstruction when vegetation is expected within the debris load for a total of 2 to 4 feet of additional width. Other documented methods may be used subject to approval by the local governing agency.

Debris control facilities require perpetual maintenance to assure proper function. Access to both debris barriers and debris basins sufficient for maintenance personnel and heavy equipment is of paramount importance. Debris basins shall have an operation and maintenance plan sheet that specifies regular inspection and maintenance at specific time intervals and physical maintenance "indicators" that let an observer know when maintenance activity is required. The operation and maintenance plan sheet shall also show how the basin is accessed from local streets. Operation and maintenance plan sheet shall ensure that vegetation and debris are removed or maintained on a regular basis to maintain the function of the facility.

The project owner and design engineer shall consult with the governing agency to determine the appropriate maintenance mechanism required for a particular project. Maintaining a debris basin can be a significant amount of work, particularly when it has filled in with sediment or debris. The District strongly recommends that all debris basins be maintained by a public entity that has the experience and dedicated manpower and equipment to perform the work. Private entities such as HOAs typically do not have the resources or experience to perform this work, and if the system is not properly maintained and fails, the downstream culvert / storm drain will not function and the protected community will be flooded.

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 sufficient to provide adequate access and maintenance. Debris basins are not required to provide 100-year route down detention attenuation, see Section 11.4 for design criteria.

Figure 11.1: Leach Debris Basin Following 2018 Holy Fire

11.3 HYDRAULIC DESIGN OF DEBRIS BARRIERS AND BASINS

11.3.1 Trash Racks

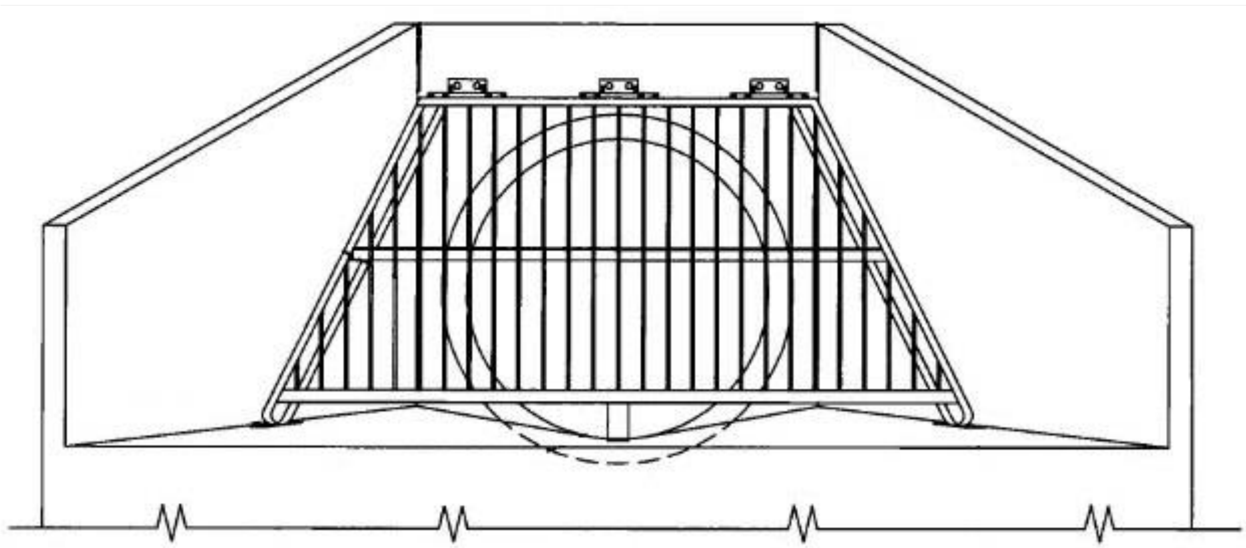
Trash racks provide a physical barrier across the upstream face of a storm drain or culvert inlet. They are applicable in areas with relatively small tributary areas and small trash/debris potential. For large areas or areas with potential for high volume of debris, a debris basin and sloped debris rack should be used as described in Section 11.4.

Trash racks vary greatly in size and materials. Many factors influence the design of trash racks, including: 1) the size and type of trash/debris, 2) the size of culvert or structure being protected, 3) the amount of flow, and 4) flooding issues.

Trash racks shall typically have an open area equivalent to approximately **two times the flow area of the conduit or channel they are protecting** to maintain flow conveyance and reduce head loss. See Section 9.7 for more information on the hydraulic analysis of trash racks.

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 pining debris where applicable. Debris trash racks shall be well-secured but removable for the purposes of maintenance. The California Standard Specifications for Public Works Construction or "Greenbook" Standard Plan Nos. 360-2 and 361-2 offers details and specifications for a sloped protection barrier and typical inclined trash (debris) rack. Maintenance access to the trash rack is critical and must be provided. Access can take the form of an easement, public trail, or public road and must include sufficient space for maintenance equipment. Where maintenance is intended from the debris rack invert, an access road and concrete paved area per Standard Drawing M827 to the debris rack must be provided. Staging and loading area shall be provided for equipment adjacent to road and outside the flow area.

Figure 11.2: Trash Rack (Inclined) Standard Plan 360-2



11.3.2 <u>Debris Posts (Bollards)</u>

A debris post is a structural system of posts (bollards) placed upstream of a culvert, storm drain, channel, or spillway entrance, causing debris to deposit before entering the structure. See Figure 11.3.

Debris posts shall be a minimum of 6 inches in diameter and are usually constructed of steel pipes filled with concrete and embedded in a concrete base. The concreted base shall be continuous along all bollards to also function as a cutoff wall/grade control structure. For culverts, 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 at least twice the culvert diameter where practicable. Bollards located in washes are typically spaced at a minimum of 3 feet from edge to edge. The maximum height of the bollard above the ground shall be 6 feet.

Debris posts shall be designed assuming the barrier is 100 percent effective in blocking the flow (plugged), the barrier will therefore act as a submerged sharp-crested weir with a height equivalent to the height of the debris posts. The freeboard between the crest of dam and basin water surface upstream of the bollards shall be a minimum of two feet. The members are designed assuming both static and dynamic loading using the mixed density of water and debris, 115 pcf. The design engineer must check that the water elevation spilling over the top of the weir will continue to flow toward the culvert and not flood the surrounding area.

Figure 11.3: Debris Posts Upstream of Spillway

Determine the Moment on the bollard assuming both static and dynamic loads.

11.3.2.1 Static Sediment/Debris

Figure 11.4: Debris Distributed Load Plan View

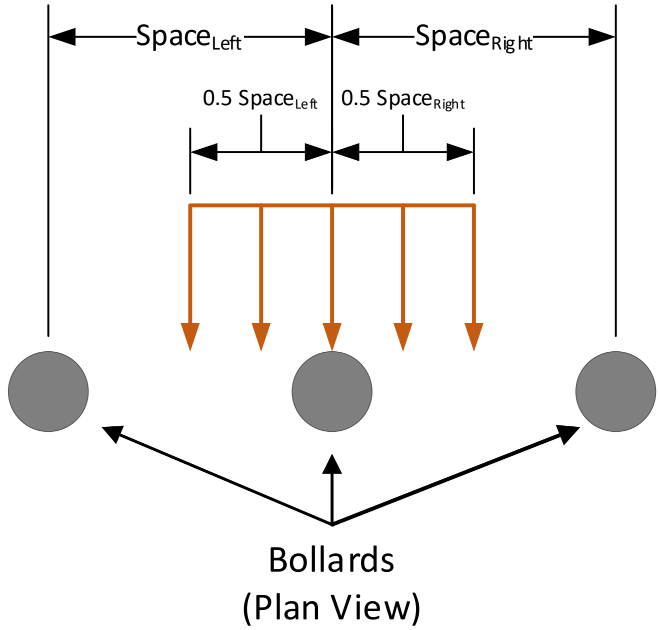
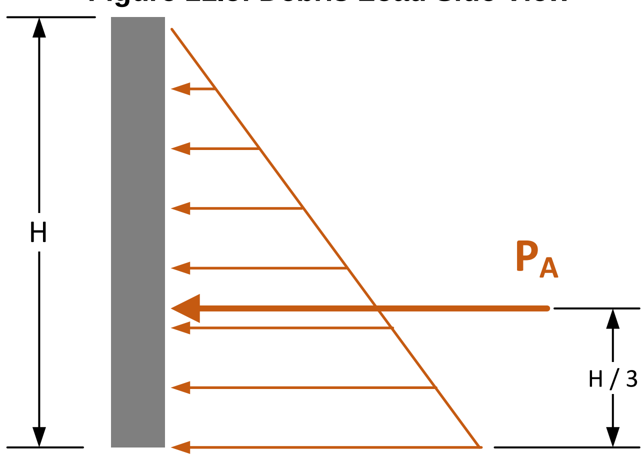


Figure 11.5: Debris Load Side View

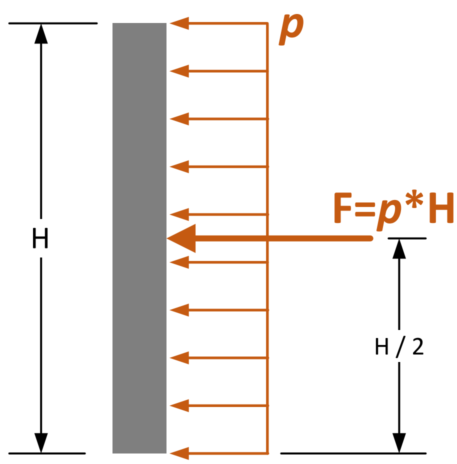


- Φ= 42° (since larger debris will also be included)
- Using Rankine active earth pressure
- Debris is a mixture of soil and water so, unit weight of debris
- Spacing between bollards is completely filled with sediment/debris
- Calculate the tributary width of debris acting on a single bollard
    a. Space is bollard width+ ½ distance to left bollard+ ½ distance to right bollard
- Calculate the debris point load (units are in lb/ft)

   a. $P_A = (1/2)K_a * \gamma_{debris} * H^2$
- Multiply value by the tributary width to get a single point load.
   a. This is because all of the trapped sediment will be loaded only on the bollards.

11.3.2.2 Sediment/Debris Loads Accounting for Velocities

Figure 11.6: Flowing Debris Load Side View



Use the AASHTO LRFD Bridge Design Specifications, Equation (11.1).

$$p = C_D \frac{w}{2g} V^2 \tag{11.1}$$

where:

p	=	pressure of flowing water (ksf)
C_D	=	drag coefficient for piers as specified in Table 11.2, assume "debris lodged against pier"
w	=	specific weight of water (kcf); instead of using the normal weight of water, it is recommended to increase the density to account for the sediment and debris = 115 lb/ft^3
g	=	acceleration due to gravity 32.2 ft/sec ^{2}
V	=	velocity of water (ft/s)

As a convenience, Equation (11.1) recognizes that $w/2g$ ≈1/1,000, but the dimensional consistency is lost in the simplification. Similar to the static loading, multiply the pressure by the spacing of the bollards, so the load is concentrated only on the bollard.

Table 11.2: Drag Coefficient

Type	C_D
Semicircular-nose pier	0.7
Square-ended pier	1.4
Debris lodged against pier	1.4
Wedged-nose pier with nose angle 90° or less	0.8

11.3.2.3 Bollard Embedment Length

Debris posts shall be embedded to a depth adequate to resist both hydrostatic pressures and dynamic pressures and help prevent failure due to scour. The Los Angeles County Flood Control District Debris Dams and Basin Design Manual (1979) provides an equation (Equation (11.2)) for embedment depth based on lateral forces that may be used.

$$L = 1.85\sqrt[3]{\frac{M_O}{R}} \tag{11.2}$$

$$M_O = \frac{M}{d}$$

where:

L	=	Length of embedment (ft)
R	=	Lateral load resistance (psf/ft) as recommended by Geotech or may use 300 (psf/ft) (accepted presumptive value per CBC §1806)
M	=	Moment applied to barrier (ft-lbs) **assuming sum of both static and dynamic loads**
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. The design engineer shall also consider the overturning moment of the entire structure including posts and concrete footing to ensure that the structure is stable under debris loads.

11.4 DEBRIS BASIN DESIGN CRITERIA

The sizing of the debris volume to be impounded by the debris basin shall be equal to the amount generated from the watershed upstream of the selected site. 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. Debris basins alone that do not account for attenuation do not require a basin routing analysis.

11.4.1 Required Volume

Methods for determining the required volume of sediment and debris are:

Sediment Only (no debris potential) - Several methods may be used to calculate sediment volumes, specifically the Flaxman Method, Corps of Engineers' *Los Angeles District Method for the Prediction of Debris Yield* (2000), and the Modified Universal Soil Loss Equation (MUSLE). The design engineer shall obtain agency approval prior to using other methods.

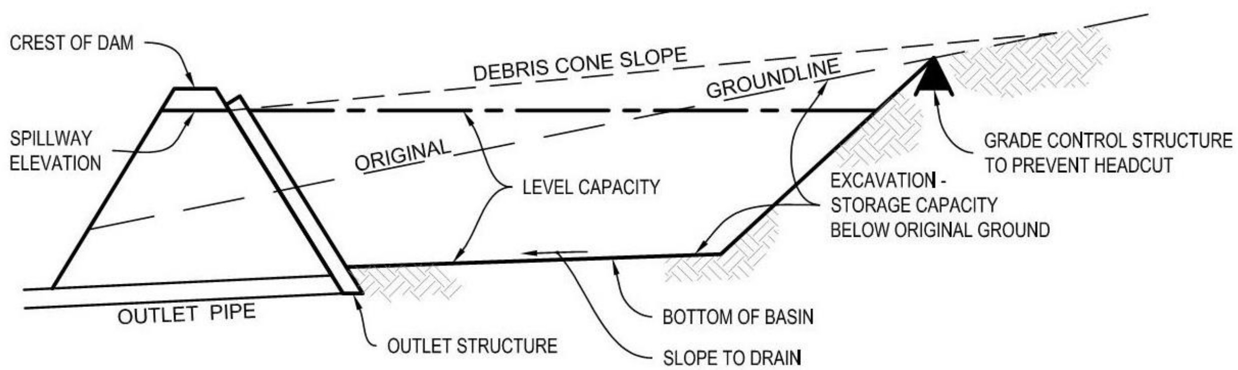
Sediment and Debris - The design engineer shall use both of the following references: Corps of Engineers' Los Angeles District Method for Prediction of Debris Yield (2000) and the Los Angeles County Department of Public Works' Sedimentation Manual 2nd Edition (2006). Because of the potential for debris basins to affect numerous properties, the design engineer shall contact the governing agency prior to beginning design to coordinate the design criteria, submittal requirements, and any legal/regulatory issues.

11.4.2 General Geometrics

The calculations for the debris volume shall be based on the assumption that the debris will be deposited in such a manner that the debris slope, sloping upstream from the spillway crest, will be equal to 50 percent of the average slope of the original stream bed for the total length of the basin site, see Figure 11.7.

Figure 11.7: Pit Type Debris Basin

(ADAPTED FROM: L.A. County DPW, 2006)



Reference Section 10.3 for other design parameters applicable to both detention and debris basins.

11.4.3 Common Criteria with Detention Basins

The following criteria described in Chapter 9 also apply to Debris Basins. See the referenced Sections as appliable:

Section Reference	Title
10.3.3	Spillways
10.3.4	Basin Side Slopes
10.3.5	Basin Floor
10.3.6	Embankment
10.3.7	Vegetation
10.3.9	Site Access

Section Reference	Title
10.3.10	Perimeter Fencing
10.3.11	Access Roads
10.3.12	Invert Maintenance Access

11.4.4 Stockpile/Staging Area

Operational areas outside the basin are required for removing, handling/drying, loading, and hauling material from a sediment/debris basin site.

1. A material stockpile/staging area shall be situated adjacent to the basin and outside of the impoundment area as approved by the Engineer.
2. Minimum stockpile footprint area shall be designed to sufficiently store 25% of the volume of debris accumulated in the 100-year frequency design event. Surface area shall be calculated by assuming a stockpile 10-feet high with 2:1 slopes.
3. A 20-feet wide access road shall be provided around the stockpile/staging area. A minimum 70-feet long and 15-feet wide area shall be provided for equipment loading and unloading outside of the stockpile/staging area and outside of traveled ways.

11.4.5 Outlet Structure

Accumulated sediment and debris will initially be saturated and waterlogged, which can make removal and hauling difficult if not impossible. An outlet structure must be provided that will allow the accumulated sediment and debris to dewater within a reasonable amount of time. Free water above the accumulated debris level must fully drain within 72 hours. The outlet structure for a debris basin proposed for District maintenance shall be the same type as required for detention basins. See Section 10.3.2.

11.5 SAFETY

Debris barriers and basins shall be designed with appropriate safety measures as described in Chapter 10 .

11.6 REFERENCES

American Public Works Association and Associated General Contractors of Southern California (APWA-AGC), 2012, California Standard Specifications for Public Works Construction 12th Edition.

County of Orange Department of Public Works, December 2020, Local Drainage Manual 2nd Edition.

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