CITY OF CORPUS CHRISTI

DRAINAGE DESIGN MANUAL

APPENDIX TO THE
INFRASTRUCTURE DESIGN MANUAL

DECEMBER 2024

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TABLE OF CONTENTS

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Chapter 1
INTRODUCTION AND PURPOSE

1.1 PURPOSE OF THE DRAINAGE DESIGN MANUAL

The primary purposes of this *Drainage Design Manual* (DDM) are:

a. To establish the minimum standards necessary to protect the safety and welfare of the public;

b. To consolidate storm water drainage criteria for all areas of the City of Corpus Christi in a single document;

c. To establish uniform criteria for the analysis, design, and construction of storm water drainage facilities in the City of Corpus Christi and its Extraterritorial Jurisdiction (ETJ); and

d. To support and illustrate the policies and criteria that are presented in the City of Corpus Christi *Infrastructure Design Manual (IDM)*.

1.2 GENERAL DRAINAGE OBJECTIVES

The City of Corpus Christi and its extraterritorial jurisdiction (ETJ) covers nine (9) drainage Basins, including:

- Nueces River
- West Oso Creek
- Oso Creek
- Nueces Bay T
- Corpus Christi Bay
- Inner Harbor
- Oso Bay
- Laguna Madre
- Padre/Mustang Island

The boundaries of these Basins are illustrated below.

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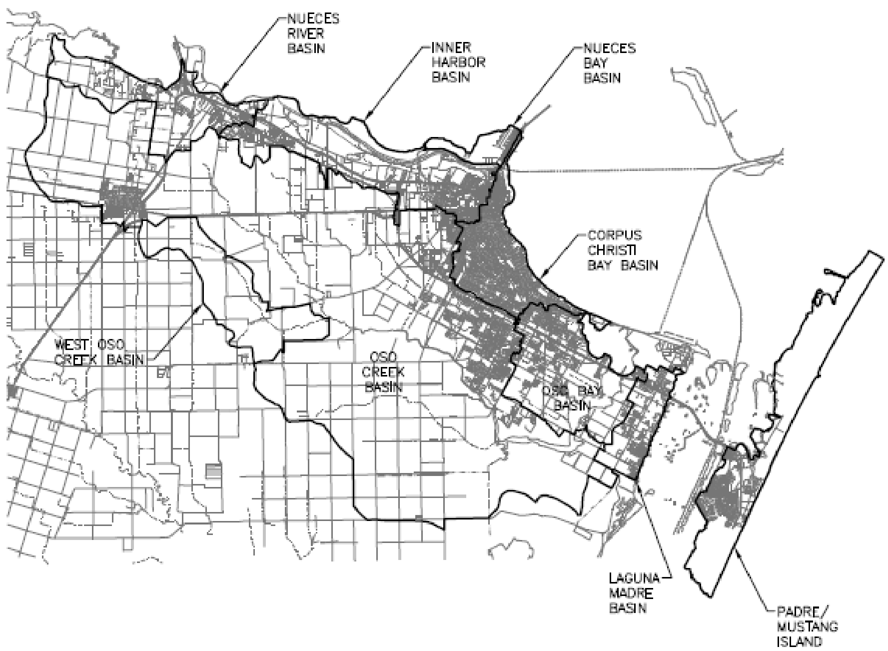
Figure 1-1. Corpus Christi Drainage Basins



General drainage objectives for the City of Corpus Christi include the following:

Flood Protection

a. Minimize potential flood damage to homes and businesses

b. Minimize adverse impacts to nearby and downstream properties

Drainage

a. Define design storms and levels of protection

b. Compliance with the *Infrastructure Design Manual* (IDM)

c. Improve storm water system design process

City/Developer Participation

a. Define Storm Water System Responsibilities

b. Promote Orderly Growth of the Storm Water System

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Maintenance

a. Incorporate Storm Water System Maintenance

b. Incorporate Procedures That Protect the Environment

Water Quality

a. Protect Community Health and Safety

b. Maintain Ecosystems and Preserve the Natural Environment

c. Storm Water Regulation Compliance

Quality of Life

a. Transform Existing Facilities into Neighborhood Assets

b. Establish and enhance Greenway Corridors

1.3 DRAINAGE DESIGN MANUAL FORMAT AND USAGE

The DDM consists of 13 Chapters and 4 Appendices. A brief introductory description of each chapter and the appendices is shown below.

1.3.1 Chapter 1 – Introduction and Purpose

Chapter 1, “Introduction and Purpose”, introduces the reader to the manual, outlines the purpose of the work, and provides an outline of the document and its features.

1.3.2 Chapter 2 – Regulatory Authority

Chapter 2, “Regulatory Authority”, provides the legal basis that empowers the City to perform the oversight and regulatory functions described throughout the manual.

1.3.3 Chapter 3 – Hydrologic & Hydraulic Concepts

Chapter 3, “Hydrologic & Hydraulic Concepts”, introduces the reader to basic concepts of hydrology and hydraulics needed to understand the more specific requirements of later chapters.

1.3.4 Chapter 4 – Hydrology

Chapter 4, “Hydrology”, includes the specific requirements needed to perform hydrologic analyses for project designs and submittals.

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1.3.5 Chapter 5 – Hydraulics

Chapter 5, “Hydraulics”, provides detailed information on the hydraulic analysis and design of drainage facilities.

1.3.6 Chapter 6 – Pavement Drainage, Roadside Ditches and Inlets

Chapter 6, “Pavement Drainage, Roadside Ditches and Inlets”, provides criteria for the design of pavement drainage, roadside ditches, and various types of inlets and inlet situations.

1.3.7 Chapter 7 – Detention Analysis

Chapter 7, “Detention”, includes criteria and guidelines to be used in the analysis and design of detention facilities.

1.3.8 Chapter 8 –Pump Stations

Chapter 8, “Pump Stations”, provides information on hydrologic and hydraulic design requirements and criteria for pump stations.

1.3.9 Chapter 9 – Water Quality

Chapter 9, “Water Quality”, includes general requirements related to water quality protection and quality of life issues.

1.3.10 Chapter 10 – Erosion and Sediment Control

Chapter 10, “Erosion and Sediment Control”, describes methods for controlling erosion and sediment deposition in drainage facilities.

1.3.11 Chapter 11 – Best Management Practices (BMPs)

Chapter 11, “Best Management Practices (BMPs)”, provides information and guidance regarding the selection, design, and use of BMPs in the construction of storm water facilities.

1.3.12 Chapter 12 – Coastal Flooding

Chapter 12, “Coastal Flooding”, provides a general overview of coastal processes, flooding, and engineering considerations including hazards, design considerations, and construction in a coastal environment.

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1.3.13 Chapter 13 – Miscellaneous Criteria

Chapter 13, "Miscellaneous Criteria", includes general criteria related to right-of-way and drainage easements, floodway and floodplain development, finished floor elevations, lot grading and drainage, and maintenance.

1.3.14 Appendices

Subject letter designated appendices supplement *DDM* Chapter information.

Appendices include:

Appendix A – Acronyms & Abbreviations
Appendix B – Glossary
Appendix C – Checklists
Appendix D – References

1.4 DRAINAGE DESIGN MANUAL PREPARATION

Pape-Dawson Engineers was tasked with reviewing and consolidating Corpus Christi's draft 2009 Drainage Design Manual (DDM) and draft 2009 Drainage Criteria Manual (DCM). The 2009 DDM and 2009 DCM were prepared by Goldston Engineering, A CH2m Hill Company, and Dodson & Associates, Inc. Neither of the draft manuals were adopted in full by the City, but some components were incorporated into the City's Infrastructure Design Manual (IDM) since 2009.

Pape-Dawson's consolidation of manuals included comparing tables of contents, chapter text, and content of appendices, then merging the 2009 DCM text into the 2009 DDM. Duplicate text was removed and chapter sections were arranged accordingly. A benchmark of technical criteria for other Texas cities was performed to compare Corpus Christi standards to other municipalities. Technical recommendations were incorporated into this consolidated manual in conjunction with the citywide storm water master plan. Chapters and appendices were reviewed, edited, added, and removed.

The three (3) chapters with the most revisions are Chapter 2 – Regulatory Authority, Chapter 7 – Detention Analysis, and Chapter 12- Coastal Flooding. Chapter 2 now includes guidance related to performing adverse impact analyses. Chapter 7 indicates where detention is required and provides guidance for what detention calculation methods shall be used. Chapter 12 is a new chapter and provides an overview of coastal engineering considerations. The remainder of the chapters were merged and edited to varying levels based on the content and context of each chapter. Figures, tables, and equations from the 2009 DDM/DWM were kept where possible. Some were modified, replaced, reformatted, relocated, or removed. Other figures, tables, and equations were added as needed.

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Chapter 2
REGULATORY AUTHORITY

The basis for City of Corpus Christi storm water management policies and supporting criteria is statutory and regulatory guidance established by federal and state authority. Chapter 2 provides the statutory and regulatory criteria that compliment this document.

2.1 FEDERAL AUTHORITY

Growing public awareness and concern for controlling water pollution led to enactment of the Federal Water Pollution Control Act Amendments of 1972. As amended in 1977, this law became commonly known as the Clean Water Act (CWA). The CWA established the basic structure for regulating discharges of pollutants into the waters of the United States. The United States Environmental Protection Agency (USEPA) regulates the provisions of the CWA in Code of Federal Regulation (CFR) Title 40 (Protection of Environment) Part 122 (EPA Administered Permit Programs: The National Pollutant Discharge Elimination System). The source document can be found at the following internet address: https://www.ecfr.gov/current/title-40/chapter-I/subchapter-D/part-122?toc=1.

The National Flood Insurance Act of 1968, as amended, authorizes the Federal Government to provide flood insurance on a national basis. Flood insurance may be sold or continued in force only in communities which enact and enforce appropriate floodplain measures. The City of Corpus Christi is a participating community.

2.2 THE NATIONAL FLOOD INSURANCE PROGRAM

The City of Corpus Christi is a participant in the National Flood Insurance Program (NFIP). This program provides affordable, federally subsidized flood insurance for homes and businesses located in flood-prone areas of cities and counties which elect to participate. The City maintains a Floodplain Administrator, whose specific position can vary based on the City' organizational structure.

The program is administered by the Federal Emergency Management Agency (FEMA), which is headquartered in Washington, D.C. Flood insurance data for participating cities and counties is published by FEMA in two formats: bound Flood Insurance Studies (FIS), and Flood Insurance Rate Maps (FIRMs). FIRMs, which provide data on 100-year (1% annual chance, or 1% a.c.) flood levels, illustrate the boundaries of the floodway, 100-year (1% a.c.) floodplain, and 500-year (0.2% a.c.) floodplain, and designate flood hazard zones for insurance purposes.

Delineations of flood-prone areas are completed in Flood Insurance Studies commissioned by individual participants (typically cities and counties) in the program. The purpose of FIS is to define areas with a certain chance of flooding. The 100-year rainfall event, which has a one- percent probability of occurring in any given year, is used as a standard measure. However, FIS and FIRMs are not intended to indicate with certainty that a particular area will or will not flood over a given period of time.

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FIS include hydrologic studies to define peak flow rates along studied streams for 10-, 50-, 100-, and 500-year rainfall events. Hydraulic analyses are also performed to establish base flood elevations (BFEs) along studied streams for each of these rainfall events and to define the boundaries of the 100- and 500-year floodplains as well as the floodway. As shown in Figure 2-1, the floodway is a corridor of effective flow that includes the channel and any adjacent land areas required to pass the 100-year peak discharge rates without increasing the water surface elevation (WSEL) at any point along the channel more than one-foot above the 100-year BFEs. FIRMs provide data on 100-year BFEs, illustrate the boundaries of the floodway and 100- and 500-year floodplains, and designate flood hazard zones for insurance purposes.

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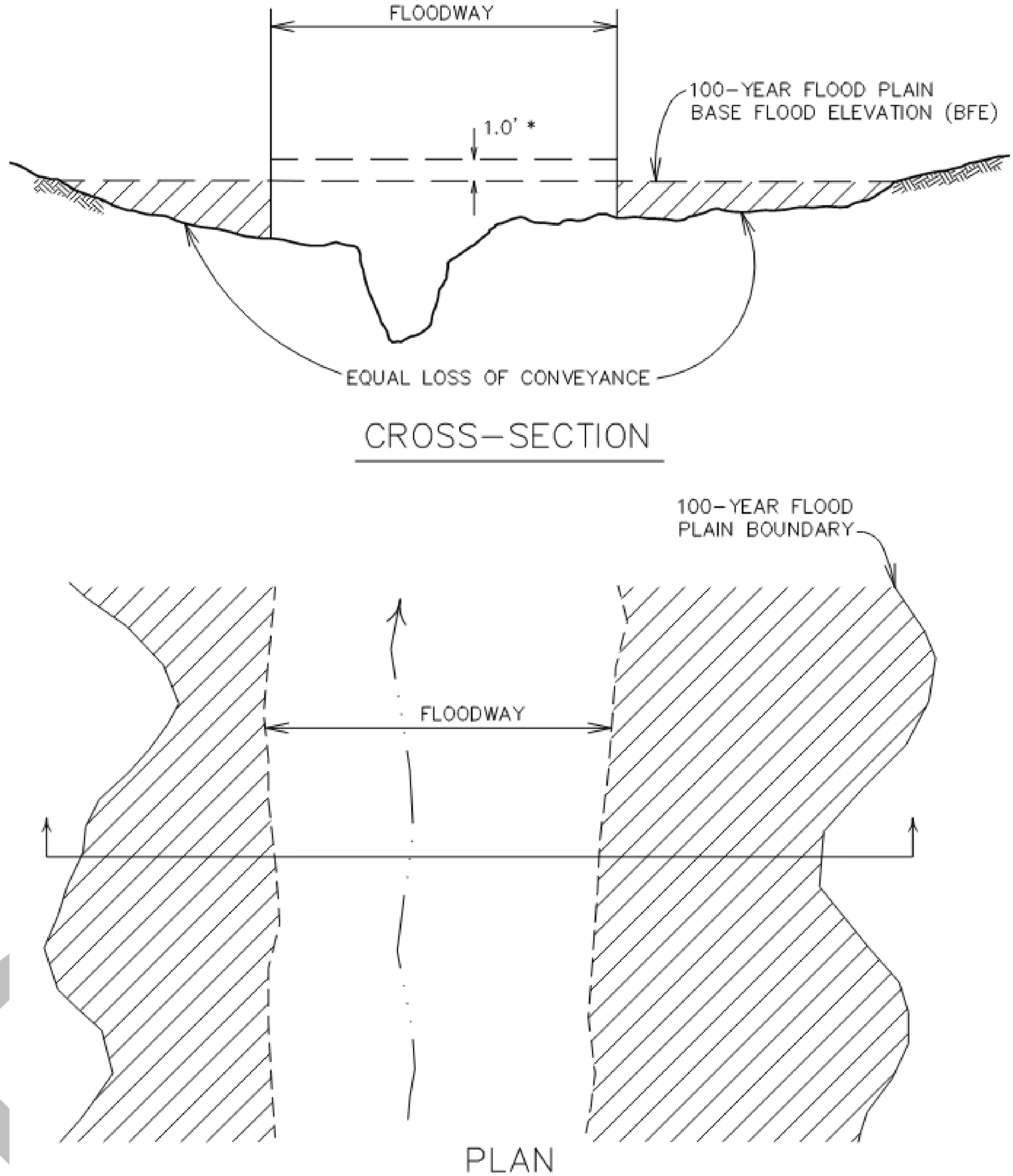
Figure 2-1. Relationship between Floodplain and Floodway



*NOTE: ANY CONSTRUCTION WITHIN THE FLOOD PLAIN CANNOT RESULT IN RAISING THE BFE MORE THAN 1.0 FT.

2.3 STATE AUTHORITY

The Texas Water Code is the legal authority that enacts the public policy of the State of Texas to provide for the conservation and development of the state's natural resources, including the control, storage, preservation, and distribution of the state's storm and floodwaters and the waters of its rivers and streams for irrigation, power, and other useful purposes.

A number of provisions made in the Texas Water Code are applicable to drainage projects falling within the jurisdiction of the City of Corpus Christi. Applicable provisions address the management of storm water runoff that prevents or avoids flooding damages to adjacent or downstream property owners and regulates the discharge of storm water runoff in compliance with the Clean Water Act. For example, Texas Water Code Section 11.086 – Overflow Caused by Diversion of Water (Chapter 11 – Water Rights) prohibits the impoundment or diversion of the natural flow of surface waters in such a manner that damages another property.

The Texas Water Code also provides for the maintenance of a proper ecological environment of the bays and estuaries of Texas and the health of related living marine resources. Specifically, Chapter 26 (Water Quality Control) prescribes the requirements for regulating discharges in concert with the Clean Water Act.

2.4 CITY AUTHORITY

The City of Corpus Christi *Drainage Design Manual (DDM) and Infrastructure Design Manual (IDM)* establish the policy and criteria for all incorporated areas of the City and its extraterritorial jurisdiction (ETJ). Any project falling within the jurisdiction of the City of Corpus Christi shall be designed and constructed in accordance with the policies and criteria presented in the *DDM and IDM*.

2.5 STORM WATER AND RELATED ORDINANCES

Ordinances and other local regulatory oversight in force for the City of Corpus Christi relating to storm water drainage include:

- City of Corpus Christi Storm Water MS4 NPDES Permit Number TXS000601
- City of Corpus Christi Unified Development Code (current adopted version)
- Code of Ordinances, City of Corpus Christi (current adopted versions):
    i. Chapter 14 (Development Services)
    ii. Chapter 42 (Platting)
    iii. Chapter 44 (Pollution Control)
    iv. Chapter 55 (Utilities)
    v. Chapter 59 (Zoning)

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2.6 CITY OVERSIGHT

The City of Corpus Christi oversees the review and approval of drainage plans through Development Services for improvements proposed by an external entity. Review of improvement designs are coordinated through Development Services with other departments as necessary.

2.7 MINIMUM LEVEL OF PROTECTION (RECURRENCE INTERVAL)

Facilities shall be designed using the Minimum Level of Protection (Recurrence Interval) as shown below and in Table 2-1. Street classifications are defined in the City of Corpus Christi's *Urban Transportation Plan*.

2.7.1 Streets and Roadways

All projects shall use the Level of Protection as shown below, as applicable to the neighboring roadways and stormwater conveyance systems.

a. Rural Roads

   i. 5-year design storm must be contained in roadside swales
   ii. 100-year design storm must not indicate ponding above the finished floor elevation of adjacent habitable structures.

b. Local/Neighborhood Streets

   i. 5-year design storm must be contained within conveyance system design and not pond across the roadway at inlets higher than the curb line
   ii. 100-year design storm must not indicate ponding above the finished floor elevation of adjacent habitable space

c. Residential Collector, Parkway Collector, and Commercial Collector Streets

   i. 5-year design storm must be contained within the conveyance system
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

d. Arterial Streets

   i. 25-year design storm must be contained within the conveyance system and roadway, and inlet design shall not pond across more than one lane
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

e. Major Highways and Freeways

   i. 50-year design storm must be contained within the conveyance system, and inlet design shall not indicate ponding across more than one lane

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ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures.

2.7.2 Storm Water Infrastructure

Storm Water conveyance facilities both above ground and underground (UG) shall be designed using the level of protection as shown below:

a. Minor Underground Storm Conveyance (<200 acres contributing area)

   i. 5-year design storm HGL at or below top of curb and MH rims
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

b. Minor Channels (<200 ac. contributing area)

   i. 5-year design storm HGL shall indicate at least 1 foot of freeboard to top of bank
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

c. Minor Channel Culverts (<200 ac. contributing area)

   i. 5-year design storm can be conveyed with no adverse impacts
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

d. Minor Channel Bridges (<200 ac. contributing area)

   i. 5-year design storm HGL shall indicate at least 1 foot of freeboard to bottom chord of bridge
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

e. Temporary Ditches (<200 ac. contributing area)

   i. 5-year design storm shall indicate at least 1 foot of freeboard to top of bank
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

f. Intermediate Underground Storm Conveyance (200ac.<contrib. area<500 ac.)

   i. 25-year design storm HGL at or below top of curb and MH rims
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

DRAFT

g. Intermediate Channels (200ac.<contrib. area<500 ac.)

   i. 25-year design storm HGL shall indicate at least 3 feet of freeboard to top of bank, with a 1-foot minimum when necessary
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

h. Intermediate Channel Culverts (200ac.<contrib. area<500 ac.)

   i. 25-year design storm can be conveyed with no adverse impacts
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

i. Intermediate Channel Bridges (200ac.<contrib. area<500 ac.)

   i. 25-year design storm HGL shall indicate at least 2 feet of freeboard to bottom chord of bridge
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

j. Major Underground Storm Water Conveyance (>500 ac. contributing area)

   i. 50-year design storm HGL at or below top of curb and MH rims
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

k. Major Channels (>500 ac. contributing area)

   i. 25-year design storm HGL shall indicate at least 3 feet of freeboard to top of bank
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

l. Major Channel Culverts (>500 ac. contributing area)

   i. 50-year design storm can be conveyed with no adverse impacts such as overtopping the driving surface
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

m. Major Channel Bridges (>500 ac. contributing area)

   i. 50-year design storm HGL shall indicate at least 1 foot of freeboard to bottom chord of bridge
   ii. 100-year design storm shall not indicate ponding above the finished floor elevation of adjacent habitable structures

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n. Storm Water Pumping Stations

   i. 100-year design storm

o. Seawalls and Associated Outlet Control Structures

   i. 100-year design storm

2.8 ADVERSE IMPACT ANALYSIS

Detention is required for projects within the Oso Creek Watershed, as discussed in Section 7.1.1. For all other watersheds, the engineer shall perform an analysis to determine whether the proposed development has an adverse impact to infrastructure or other properties.

Engineers shall analyze hydraulic capacity downstream from the development under existing, proposed, and ultimate (full build out of the entire watershed) conditions. The following criteria will be used to determine how far downstream must be analyzed to determine if there is an adverse impact to the receiving storm water infrastructure or other properties and structures.

Hydraulic analyses will be performed from the downstream edge of the development to the closest point where one of the following applies:

a. The receiving watershed is ten times (10x) larger than the area of the development.

b. For storm drain systems, where the storm drain outfalls into an open channel that has adequate capacity for the receiving watershed.

c. For streets, to the closest storm drain system with adequate capacity for the receiving watershed.

Proposed and ultimate conditions will be compared to existing conditions for design storm events included in Section 2.7, based on the type of infrastructure (street, storm drain, channel, etc.). Engineer is responsible for calculating flows leaving the site to be developed and the flows in the receiving system(s). Flows will be calculated for all storm events pertinent to the level of protection. See the examples at the end of this chapter. Careful attention must be paid to ongoing development within the watershed of the project site. Coordinate with the City regarding nearby developments and capital projects, and how those should be taken into account during the adverse impact analysis.

If the proposed development results in increases in WSE at a building (residential, non-residential, commercial, industrial, etc.) or if the proposed development results in receiving infrastructure being over capacity, then the increase is considered an adverse impact. If the receiving infrastructure is over capacity in existing conditions, then proposed peak flows may not result in further increases to the WSE. If it is determined that there is no adverse impact, then detention will not be required. If the development does have an adverse impact, then the development must include detention or other infrastructure improvements to mitigate increased WSE. If the City and engineer agree to downstream infrastructure improvements in lieu of detention, the

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downstream infrastructure improvements shall be sized to convey ultimate condition flows for the entire watershed.

2.8.1 Example Adverse Impact Analysis Scenarios

Example 1

Scenario: A 2-acre site drains by surface flow to an adjacent collector street, then into a minor underground storm drain system, and eventually to a channel with a drainage area ≥20 acres.

Limits of Analysis: Analysis would extend to a point where the cumulative drainage area is ≥20 acres (2 x 10), or where the street drains into a storm drain with adequate capacity, whichever is closer.

Level of Protection to Analyze: **Collector Street and Minor Underground Storm.** Confirm the proposed 5-year storm is contained within the top of curb, the 5-year storm HGL is contained below the top of curb (at inlets) and below manhole rims, and the proposed 100-year storm does not create or worsen ponding/flow at or above downstream finished floor elevations. (If adequate capacity, no detention is required. If inadequate capacity, detention is required.)

Example 2

Scenario: A 30-acre site drains by minor underground storm drain into an intermediate channel that eventually reaches Corpus Christi Bay.

Limits of Analysis: Analysis would extend to a point where the cumulative drainage area is ≥300 acres (30 x 10), or where the underground system drains into a channel with adequate capacity, whichever is closer.

Level of Protection to Analyze: **Minor Underground Storm.** Confirm the proposed 5-year storm HGL is contained below the top of curb (at inlets) and below manhole rims, and the proposed 100-year storm does not create or worsen ponding/flow at or above downstream finished floor elevations. **Intermediate Channel.** Confirm the proposed design storm HGL indicates at least 1' freeboard at top of bank, and the proposed 100-year storm does not create or worsen ponding/flow at or above downstream finished floor elevations. (If adequate capacity, no detention is required. If inadequate capacity, detention is required.)

Example 3

Scenario: A 10-acre site drains by minor channel with cross culverts, then into a major channel.

Limits of Analysis: Analysis would extend to a point along the minor channel where the cumulative drainage area is ≥100 acres (10 x 10), or where the minor channel drains into

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the major channel (drainage area >500 acres), whichever is closer.

Level of Protection to Analyze: **Minor Channel and Culverts.** Confirm the proposed 5-year storm is contained within the channel with at least 1' freeboard at top of bank and that the 5-year storm can be conveyed through culverts without overtopping. Confirm the proposed 100-year storm does not create or worsen ponding/flow at or above downstream finished floor elevations. (If adequate capacity, no detention is required. If inadequate capacity, detention is required.) Analysis of major channel not required since drainage area > 10x the area of the site.

<u>Example 4</u>

Scenario: A 550-acre site drains by existing constructed major channel, then to a natural major channel with a bridge, then into the Nueces River.

Limits of Analysis: Analysis would extend to a point along the major channels to a point where the cumulative drainage area is ≥5,500 acres, or to the Nueces River, whichever is closer.

Level of Protection to Analyze: **Major Channels and Bridges.** Confirm the 25-year design storm HGL indicates at least 3' freeboard to top of channel banks, and the 50-year design storm HGL indicates at least 1' freeboard to the bottom chord of the bridge. Confirm the proposed 100-year storm does not create or worsen ponding/flow at or above downstream finished floor elevations. Analysis of the Nueces river is not required since drainage area > 10x the area of the site.

<u>Example 5</u>

Scenario: A 2-acre site drains directly into the Nueces River or Corpus Christi Bay.

Limits of Analysis: Downstream analysis not required since the receiving drainage area is much greater than 10x the area of the site. Ensure site design does not divert runoff to adjacent properties.

Level of Protection to Analyze: Downstream analysis not required since the receiving drainage area is much greater than 10x the area of the site. Ensure site design does not divert runoff to adjacent properties.

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Chapter 3
HYDROLOGIC & HYDRAULIC CONCEPTS

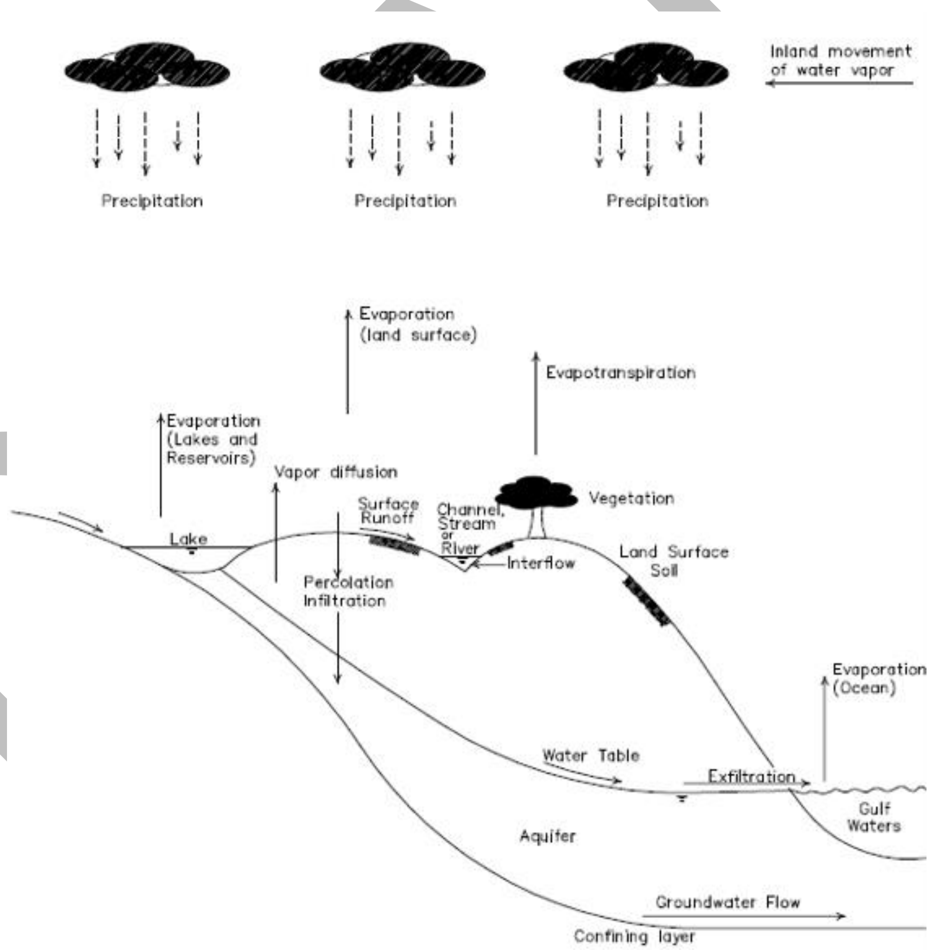
The purpose of this chapter is to present a brief summary of hydrologic and hydraulic concepts that are required to understand and apply the criteria presented in the *DDM and IDM*. This chapter also includes a description of the effects of urbanization on the watershed as well as a description of the National Flood Insurance Program.

3.1 BASIC HYDROLOGIC CONCEPTS

3.1.1 The Hydrologic Cycle

The term hydrologic cycle refers to a series of processes through which moisture falls to earth as precipitation and returns to the atmosphere. The basic processes involved in the hydrologic cycle include rainfall, infiltration, interflow, storage, evaporation, and transpiration. Figure 3- 1 illustrates the interaction of these processes.

Figure 3-1. Hydrologic Cycle Diagram



3.1.2 Design Rainfall Events

Rainfall normally occurs in irregular patterns with respect both to space and time. However, synthetic rainfall events (referred to as "design storm events") are typically used for hydrologic analyses. These design storm events are developed through statistical analyses of long periods of recorded rainfall data and are defined by the recurrence interval and storm duration. For example, a 100-year, 24-hour storm is a 24-hour duration design storm which has a one-percent probability of occurring in any given year.

3.1.3 Infiltration & Runoff

A portion of the rainfall that reaches the earth soaks into the ground via infiltration, while the balance of the rainfall is called runoff (Figure 3-1). Since infiltration increases with the porosity of the soil, infiltration for clay soils is less than for sandy soils. Infiltration is reduced as the moisture content of the soil is increased and ceases when the soil becomes saturated. As infiltration decreases, runoff increases and vice versa.

3.1.4 Runoff Hydrographs

Runoff hydrographs are relationships between the rate of runoff and time. Hydrographs are important because they provide information on the peak rate of runoff and variations in runoff rates throughout the duration of a particular storm event. These variations can be significant in defining the response of a watershed to a rainfall event, especially when the watershed is large and runoff continues over many hours or days.

A unit hydrograph is a hydrograph which reflects the response of a watershed to a rainfall event that produces exactly one-inch of runoff. Runoff hydrographs for storm events producing more or less than one-inch of runoff are computed from a unit hydrograph by multiplying each individual flow rate in the unit hydrograph by the actual runoff volume in inches. This computation is based on various hydrologic parameters and is performed automatically by software programs such as HEC-HMS, which was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers (USACE). Figure 3-2 illustrates the relationship between unit hydrographs and runoff hydrographs.

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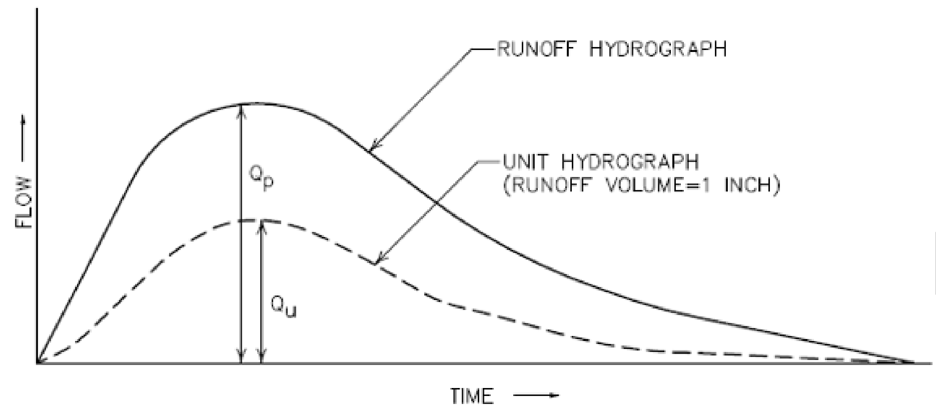
Figure 3-2. Runoff Hydrograph vs. Unit Hydrograph



$$Q_p = Q_u \times R$$

WHERE: Q_p = FLOW RATE FOR RUNOFF HYDROGRAPH
Q_u = FLOW RATE FOR UNIT HYDROGRAPH
R = VOLUME OF RUNOFF UNDER RUNOFF HYDROGRAPH

3.2 BASIC HYDRAULIC CONCEPTS

3.2.1 Manning’s Equation

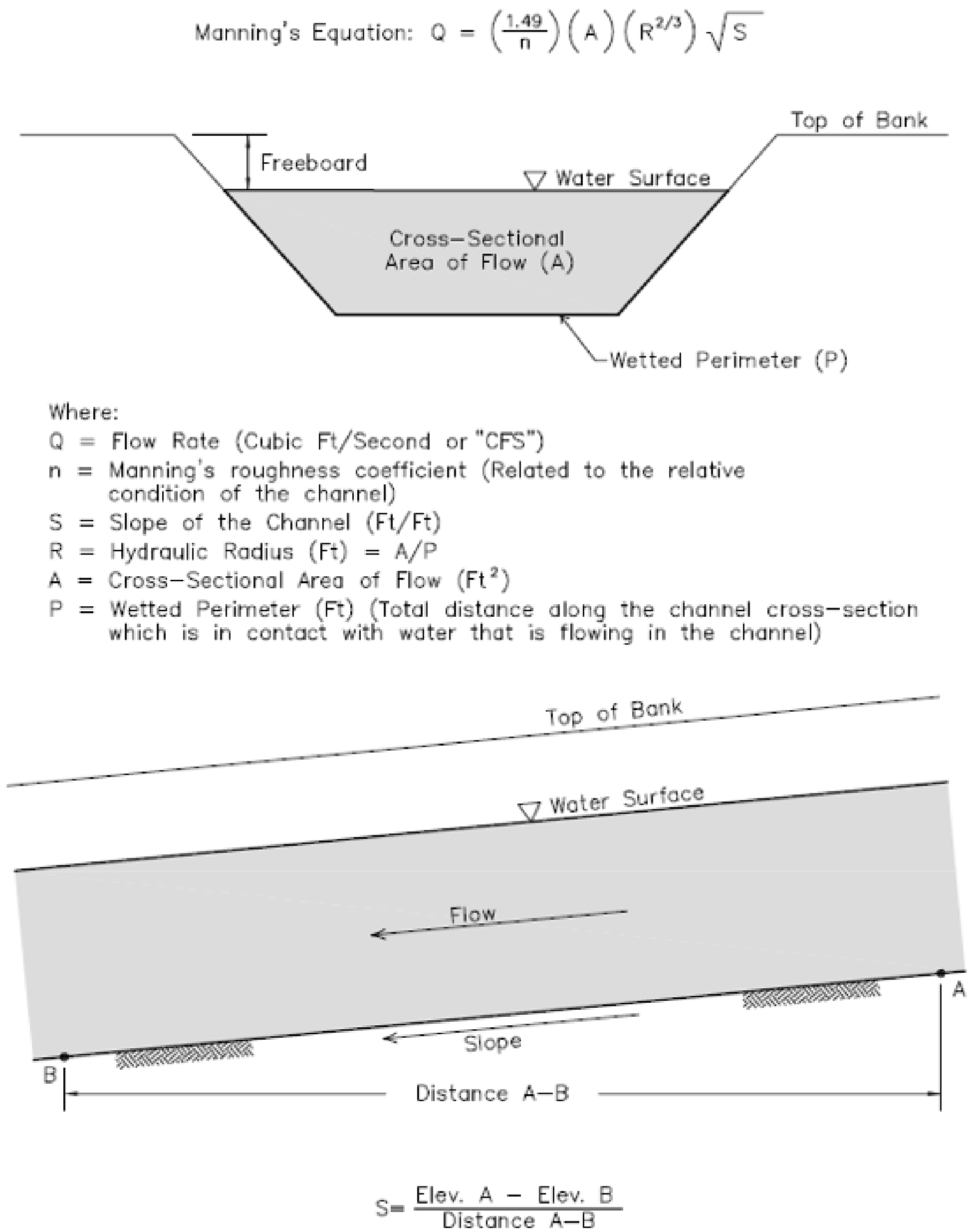
Manning’s equation is a commonly used formula that relates the hydraulic capacity and the physical condition of an open channel, a storm sewer pipe, or a box culvert. The equation is written as follows:

$$Q = \left(\frac{1.49}{n}\right) AR^{2/3}S^{1/2}$$ **Equation 3-1**

where: Q = flow rate (cubic feet per second);
n = Manning’s roughness coefficient;
A = cross-sectional area of flow (square feet);
R = hydraulic radius, cross-sectional area divided by wetted perimeter (feet); and,
S = slope of the pipe or channel (feet per foot).

Application of this equation is discussed in detail in Chapter 5. The roughness coefficient (*n* value) is a measure of the roughness of the surfaces with which water comes into contact; higher *n* values represent rougher surfaces and lower *n* values represent smoother surfaces. Information on selecting *n* values for open channels and storm sewers is included in Chapter. Figure 3-3 illustrates some of the basic concepts associated with Manning's Equation.

Figure 3-3. Hydrologic Cycle Diagram



3.2.2 Conveyance

Conveyance is a measure of the capacity of a channel, floodplain, or hydraulic structure to carry storm water. As indicated in Equation 3-2, conveyance increases with the cross-sectional area of flow, the depth of flow in the structure, and the smoothness of the surfaces with which water comes into contact. For example, enlarging a drainage channel will increase the conveyance and the rate of storm water flow within the channel. Clearing away trees and brush from a channel may have the same effect. Replacing a corrugated metal pipe (CMP) with a reinforced concrete pipe (RCP) of the same diameter may also result in an increased conveyance because of the smoother interior of the RCP.

$$K = \left(\frac{1.49}{n}\right) AR^{2/3}$$ **Equation 3-2**

where: K = conveyance (cubic feet per second).

3.3 EFFECTS OF URBANIZATION

Urbanization includes activities such as land clearing, new development, roadway construction, improvements to drainage systems, changes in natural land topography, placement of fill in floodplains, and construction of pavements and other impervious surfaces. These types of activities may have significant effects on the response of a watershed to rainfall, which are summarized below.

- **Increased Volume of Runoff:** Urbanization is typically accompanied by an increase in the percentage of the ground surface that is covered by impervious materials, which decreases infiltration and increases the volume of runoff.

- **Increased Rate of Runoff:** In most urbanized areas, drainage systems are designed to collect and convey storm water as efficiently as possible away from areas occupied by homes, businesses, and roadways. This efficiency tends to concentrate storm water runoff more quickly than the natural drainage system in most areas. In addition, re-grading of natural slopes and the removal of flow-retarding vegetation eliminates natural storage that attenuates runoff rates in non-urbanized areas. These factors cause runoff rates from urbanized areas to exceed rates from undeveloped areas, which tends to increase the water surface elevations (WSEs) in channels.

- **Modified Watershed Response:** The increased efficiency of urban drainage systems tends to decrease the time of concentration from developed drainage areas so that the peak runoff rate occurs more quickly than from the same area prior to development. As a result, development of a drainage area may adversely impact WSEs within the receiving channel due to changes in the timing of peak runoff rates. These adverse impacts may occur even if detention is provided and the developed peak runoff rate is less than the undeveloped peak runoff rate.

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- **Reduced Floodplain Conveyance:** Lots and/or building pads located in flood-prone areas are typically elevated with fill material. The placement of this material in floodplains creates obstructions to flow and reduces the available conveyance in the floodplain. The construction of elevated roads across the floodplain has a similar effect. Such reductions in the conveyance capacity of the floodplain tend to increase WSEs in channels.

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Chapter 4
HYDROLOGY

The purpose of this chapter is to provide detailed information on the hydrologic analyses required by the City of Corpus Christi. This chapter is divided into two main sections. Section 4.1 describes requirements for the hydrologic analysis of drainage areas up to 200 acres, while Section 4.2 describes requirements for the hydrologic analysis of drainage areas greater than 200 acres. These hydrologic methods shall be used for calculating runoff under existing, proposed, and ultimate conditions.

4.1 DRAINAGE AREAS UP TO 200 ACRES

4.1.1 Introduction

This section describes the methods to be used in hydrologic analyses of drainage areas up to 200 acres. These analyses may be completed using the Rational Method.

4.1.2 The Rational Method

The Rational Method relates the runoff rate from a watershed to drainage area, land use, and rainfall intensity. The basic equation used in the Rational Method to compute the runoff rate is:

$$Q = \mathrm{CIA}$$ **Equation 4-1**

where: Q = the peak runoff rate (cubic feet per second);
C = runoff coefficient dependent on land use;
I = the rainfall intensity (inches per hour); and
A = the drainage area (acres).

4.1.3 Establishing the Drainage Area

Drainage areas for Rational Method analyses shall be established by the design engineer using topographic survey, LiDAR data, storm sewer and channel layouts, and other available information. When establishing the drainage area, the contributing acreage upstream that runs into and/or through a development shall be included. Design engineer is responsible for delineating drainage areas for receiving storm drain systems, ditches, channels, streams, culverts, bridges, etc.

4.1.4 Determining Runoff Coefficients

The runoff coefficient (C) is dependent on land use, soil type, and overland slope, and shall be determined based on Table 4-1. In the event that the ultimate land use is less intense than the existing land use, then a lower runoff coefficient value may be used if supported by engineering calculations.

The hydrologic soil group may be determined from the Soil Survey for Nueces County, Texas or the Soil Survey Geographic Database (SSURGO) for Nueces County.

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For drainage areas with multiple land uses, runoff coefficients and drainage areas associated with each land use shall be determined and the composite runoff coefficient computed using Equation 4-2:

$$C_w = \frac{\Sigma(C_i A_i)}{A_T}$$ **Equation 4-2**

where: C_w = weighted runoff coefficient;
C_i = runoff coefficients for various land uses;
A_i = drainage areas corresponding to values of C_i (acres); and,
A_T = total drainage area (acres).

Table 4-1 provides a summary of runoff coefficients for various land uses, overland slopes, and soil types. The appropriate runoff coefficient may be selected by establishing the land use and consulting this table. Land use data may be obtained from zoning maps, aerial photographs, and site visits.

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Table 4-1. Rational Method Coefficients

Land Use	**Zoning District**	**Overland Slope**		
	Proposed (*Existing*)	**< 1%**	**1% - 3.5%**	**> 3.5%**
Undeveloped Land:				
• *Pasture:*				
Sandy Soil (soil groups A & B)	F-R	0.25	0.35	0.40
Clay Soil (soil groups C & D)	F-R	0.30	0.40	0.50
• *Cultivated:*				
Sandy Soil (soil groups A & B)	F-R	0.30	0.55	0.70
Clay Soil (soil groups C & D)	F-R	0.35	0.60	0.80
• *Lawn Areas:*				
Sandy Soil (soil groups A & B)		0.05	0.08	0.12
Clay Soil (soil groups C & D)		0.15	0.18	0.22
• *Woodlands:*				
Sandy Soil (soil groups A & B)		0.15	0.18	0.25
Clay Soil (soil groups C & D)		0.18	0.20	0.30
Residential Estate District	RE	0.30	0.35	0.40
One-Family Residential Districts:				
• *Lots greater than 1/3 acre*	RS-1 (*RA*)	0.30	0.40	0.50
• *Lots 1/4 to 1/3 acre*	RS-2 (*R-1A*)	0.40	0.50	0.60
• *Lots less than 1/4 acre*	RS-2, RS-3, RS-4	0.50	0.55	0.60
Townhouse Dwelling Districts	R-TH	0.60	0.65	0.70
Multiple Dwelling Districts	R-TF (*R-2*)	0.60	0.65	0.70
Travel Trailer Park District	R-MH (*T-1A*)	0.60	0.65	0.70
Manufactured Home Park District	R-MH (*T-1B*)	0.60	0.65	0.70
Manufactured Home Subdivision District	R-MH (*T-1C*)	0.60	0.65	0.70
Apartment House District	RM-1, RM-2, RM-3	0.75	0.80	0.85
Apartment Tourist District	RM-3 (*AT*)	0.75	0.80	0.85
Business Districts:				
• *Professional Office District*	OF (*AB*)	0.75	0.80	0.85
• *Neighborhood Business District*	CN (*B-1, B-1A*)	0.75	0.80	0.85
• *CC Beach Design District*	CR (*BD*)	0.75	0.80	0.85
• *Barrier Island Business District*	CR (*B-2A*)	0.75	0.80	0.85
• *Bayfront Business District*	CR (*B-2*)	0.75	0.80	0.85
• *Business District*	CG (*B-3*)	0.75	0.80	0.85
• *General Business District*	CG (*B-4*)	0.75	0.80	0.85
• *Primary Business District*	CI (*B-5*)	0.85	0.85	0.85
• *Primary Business Core District*	CBD (*B-6*)	0.85	0.85	0.85
Industrial Districts:				
• *Limited Industrial District*	BP (*I-1*)	0.50	0.65	0.80
• *Light Industrial District*	IL (*I-2*)	0.50	0.65	0.80
• *Heavy Industrial District*	IH (*I-3*)	0.60	0.75	0.85
Railroad Yard Areas		0.20	0.30	0.40
Parks, Greenbelts, Cemeteries		0.25	0.35	0.40
Playgrounds		0.20	0.28	0.35
Streets:				
• *Asphalt*		0.80	0.80	0.80
• *Concrete (Streets, driveways, and sidewalk)*		0.85	0.85	0.85
Roofs		0.85	0.85	0.85

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4.1.5 Establishing the Time of Concentration for Drainage Areas up to 200 Acres

The time of concentration (T_c) is defined as the time (in minutes) required for all portions of the watershed to contribute runoff at the computation point. The T_c is normally calculated by identifying the longest flow path within the watershed and estimating the time required for runoff to travel the entire length of this path. Storm water runoff may pass through a range of flow conditions as it moves along the longest flow path. Overland sheet flow is characterized by very shallow depths of less than two inches. Within a short distance of about 100 to 300 feet, storm water runoff begins to flow at greater depths and to collect in streets, swales, and small ditches or gullies, and is commonly known as concentrated overland flow. Finally, the runoff collects in storm sewers, creeks, and drainage channels in which flow depths may reach several feet.

In order to estimate T_c, the longest flow path is divided into reaches that represent the various types of flow conditions and the flow velocity for each individual reach is estimated. For example, the longest flow path may include overland sheet flow, concentrated flow in a roadside ditch, and flow in a drainage channel. Flow velocities for overland sheet flow and some concentrated flow conditions may be estimated using the Uplands Method, which relates flow velocity to overland slope and land use and was developed by the U.S. Department of Agriculture Natural Resources Conservation Service (NRCS), formerly Soil Conservation Service (SCS).

For storm sewers, creeks, and channels, flow velocities may be estimated using Manning's equation or HEC-RAS or other software models (see Chapter 5). The length of each individual reach is divided by the flow velocity to obtain the time of travel required for water to pass through the reach, and the T_c is equal to the sum of the individual times of travel. It is important to note that the T_c methods discussed in Section 4.2.4 shall be used for all drainage areas greater than 200 acres that require a unit hydrograph analysis.

The total time of concentration is often driven by the overland sheet flow time of concentration. Sheet flow time of concentration must be calculated based on watershed conditions. The maximum overland sheet flow time of concentration for single family residential development is 30 minutes.

The maximum overland sheet flow time of concentration for commercial area development is 15 minutes.

The time of concentration is equal to the sum of the individual times of travel (overland flow,concentrated flow, and channelized flow).

4.1.6 Computation of the Rainfall Intensity

The rainfall intensity (I) used in the Rational Method may be determined from the Nueces County intensity-duration-frequency (IDF) data from *NOAA Atlas 14 Precipitation-Frequency Analysis of the United States, Volume 11 Version 2.0: Texas,* published by the

DRAFT

National Oceanic and Atmospheric Administration (NOAA) in 2018. Related information can be found online at: https://hdsc.nws.noaa.gov/pfds. This data provides rainfall intensities for 1-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, 500, and 1000-year storm frequencies.

For TxDOT facilities, the rainfall intensity used in the Rational Method shall be calculated based on Equation 4-3, which was developed by TxDOT from TP-40 and HYDRO-35.

$$I = \frac{b}{(T_c + d)^e}$$ **Equation 4-3**

where: T_c = time of concentration (minutes); and,
b, d and e = empirical factors that characterize the IDF curves for Nueces County.

Reference the TxDOT Hydraulic Design Manual for the latest b, d, and e values.

4.1.7 Analyzing a Watershed with Multiple Sub-Areas or Computation Points

When analyzing a watershed with multiple sub-areas or computation points, the peak flow rate at the computation point located furthest upstream is computed first. Peak flow rates are computed at subsequent points, while moving in the downstream direction. At each point, the total drainage area is determined, and T_c is computed for the longest flow path from the most remote point in the entire watershed to the current computation point. The rainfall intensity for the peak flow rate computation is computed using this T_c. As discussed in Section 4.1.4, a weighted runoff coefficient shall be computed using the coefficients for individual sub-areas upstream of the computation point.

4.2 DRAINAGE AREAS GREATER THAN 200 ACRES

This section describes methods to be used in hydrologic analyses of drainage areas greater than 200 acres. These analyses shall be completed using the HEC-HMS computer program developed at the Hydrologic Engineering Center of the U.S. Army Corps of Engineers (USACE), or other appropriate software. This software program can be downloaded from the USACE's website (http://www.hec.usace.army.mil/software/hec-hms/) at no charge. The *Hydrologic Modeling System HEC-HMS User's Manual,* the *Hydrologic Modeling System HEC-HMS Applications Guide,* and the *Hydrologic Modeling System HEC-HMS Technical Reference Manual* developed by the USACE can be found for further reference. These manuals can also be downloaded from the USACE's website. The hydrologic parameters discussed in Sections 4.2.1 to 4.2.5 are the basis for developing HEC-HMS models.

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4.2.1 Watershed Boundaries

LIDAR data and other topographic information shall be used to provide the level of detail necessary to delineate additional watershed boundaries as needed.

The number of sub-areas required for the HEC-HMS analysis is a function of the number of computation points, which are typically established at confluences with tributaries, roadway crossings, or other points of interest (ponds, rivers, etc.). Normally, there is one sub-area above the first analysis point and one or more between each pair of successive analysis points. In addition, there is at least one sub-area for each tributary.

4.2.2 Rainfall Data

Rainfall depth-duration-frequency data for the City of Corpus Christi shown in Table 4-3, was obtained from the NOAA Atlas 14 website. This particular table is based on Station 79-0048, located at the Corpus Christi International Airport. Site-specific data may be obtained at the Atlas 14 website. The rainfall depth data and exceedance probability associated with the design storm event shall be entered in the meteorological model of HEC-HMS as part of the hydrologic analysis. For additional information on the required design storm events for various drainage areas and system components, refer to Chapter 2 and Chapter 5.

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Table 4-2. Rainfall Depth for a Given Duration, (Station ID: 79-0048, Corpus Christi)

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.482 (0.365-0.637)	0.566 (0.429-0.735)	0.697 (0.530-0.914)	0.810 (0.608-1.08)	0.970 (0.708-1.34)	1.10 (0.781-1.56)	1.23 (0.849-1.78)	1.36 (0.915-2.03)	1.54 (0.996-2.36)	1.66 (1.05-2.63)
10-min	0.766 (0.580-1.01)	0.900 (0.683-1.17)	1.11 (0.844-1.46)	1.29 (0.969-1.72)	1.55 (1.13-2.14)	1.76 (1.25-2.49)	1.97 (1.36-2.86)	2.17 (1.46-3.23)	2.43 (1.58-3.74)	2.62 (1.66-4.14)
15-min	0.971 (0.735-1.28)	1.14 (0.862-1.48)	1.40 (1.06-1.83)	1.62 (1.22-2.16)	1.94 (1.41-2.67)	2.19 (1.56-3.10)	2.45 (1.69-3.55)	2.71 (1.82-4.03)	3.04 (1.98-4.69)	3.30 (2.09-5.21)
30-min	1.38 (1.04-1.82)	1.61 (1.22-2.09)	1.97 (1.50-2.58)	2.28 (1.71-3.03)	2.71 (1.98-3.74)	3.06 (2.17-4.33)	3.42 (2.36-4.96)	3.79 (2.54-5.63)	4.27 (2.77-6.58)	4.64 (2.94-7.34)
60-min	1.80 (1.36-2.38)	2.11 (1.60-2.74)	2.60 (1.98-3.41)	3.02 (2.27-4.03)	3.62 (2.64-4.98)	4.10 (2.90-5.79)	4.60 (3.17-6.66)	5.12 (3.44-7.61)	5.83 (3.78-8.98)	6.38 (4.04-10.1)
2-hr	2.18 (1.66-2.86)	2.61 (1.99-3.35)	3.28 (2.50-4.26)	3.86 (2.91-5.10)	4.70 (3.44-6.41)	5.39 (3.84-7.55)	6.13 (4.24-8.79)	6.92 (4.67-10.2)	8.04 (5.24-12.3)	8.95 (5.68-14.0)
3-hr	2.39 (1.83-3.12)	2.91 (2.21-3.69)	3.69 (2.83-4.77)	4.39 (3.32-5.78)	5.42 (3.98-7.34)	6.26 (4.47-8.71)	7.17 (4.98-10.2)	8.18 (5.53-12.0)	9.63 (6.29-14.6)	10.8 (6.88-16.7)
6-hr	2.75 (2.11-3.56)	3.42 (2.60-4.29)	4.42 (3.40-5.66)	5.33 (4.06-6.96)	6.68 (4.93-8.98)	7.81 (5.60-10.8)	9.05 (6.32-12.8)	10.5 (7.09-15.1)	12.5 (8.18-18.7)	14.2 (9.06-21.7)
12-hr	3.13 (2.42-4.02)	3.96 (3.02-4.89)	5.18 (4.01-6.58)	6.31 (4.83-8.17)	8.01 (5.94-10.7)	9.44 (6.81-12.9)	11.0 (7.74-15.4)	12.9 (8.77-18.4)	15.6 (10.2-23.0)	17.8 (11.4-26.9)
24-hr	3.50 (2.72-4.46)	4.51 (3.44-5.50)	5.97 (4.64-7.51)	7.34 (5.65-9.42)	9.40 (7.02-12.4)	11.2 (8.10-15.1)	13.1 (9.26-18.2)	15.4 (10.5-21.7)	18.7 (12.4-27.3)	21.5 (13.8-32.0)
2-day	3.81 (2.98-4.81)	5.02 (3.84-6.04)	6.75 (5.28-8.42)	8.39 (6.49-10.7)	10.9 (8.17-14.3)	13.0 (9.49-17.5)	15.4 (10.9-21.0)	18.0 (12.4-25.1)	21.8 (14.5-31.4)	24.9 (16.1-36.7)
3-day	4.05 (3.18-5.09)	5.36 (4.13-6.44)	7.27 (5.70-9.02)	9.05 (7.02-11.4)	11.7 (8.84-15.3)	14.0 (10.3-18.7)	16.5 (11.7-22.5)	19.3 (13.3-26.8)	23.3 (15.5-33.3)	26.5 (17.2-38.7)
4-day	4.32 (3.39-5.41)	5.67 (4.39-6.82)	7.68 (6.04-9.50)	9.52 (7.40-12.0)	12.3 (9.26-15.9)	14.6 (10.7-19.4)	17.2 (12.2-23.2)	20.0 (13.8-27.6)	24.0 (16.0-34.1)	27.3 (17.7-39.6)
7-day	5.03 (3.97-6.25)	6.44 (5.04-7.76)	8.57 (6.78-10.6)	10.5 (8.19-13.1)	13.3 (10.1-17.1)	15.6 (11.5-20.5)	18.2 (13.0-24.4)	21.0 (14.5-28.7)	25.0 (16.7-35.3)	28.4 (18.5-40.8)
10-day	5.58 (4.42-6.92)	7.04 (5.56-8.50)	9.28 (7.37-11.4)	11.3 (8.83-14.0)	14.1 (10.7-18.1)	16.5 (12.1-21.5)	19.0 (13.6-25.3)	21.8 (15.2-29.7)	25.9 (17.4-36.3)	29.3 (19.1-41.8)
20-day	6.98 (5.56-8.58)	8.64 (6.91-10.4)	11.3 (9.02-13.7)	13.5 (10.6-16.7)	16.7 (12.7-21.0)	19.1 (14.1-24.7)	21.7 (15.6-28.6)	24.6 (17.2-33.0)	28.6 (19.2-39.5)	31.9 (20.8-44.8)
30-day	8.12 (6.48-9.92)	9.93 (8.01-12.0)	12.9 (10.4-15.7)	15.3 (12.1-18.9)	18.7 (14.3-23.5)	21.3 (15.8-27.3)	24.0 (17.3-31.4)	26.9 (18.8-35.9)	30.9 (20.8-42.4)	34.1 (22.3-47.6)
45-day	9.80 (7.85-11.9)	11.8 (9.61-14.3)	15.2 (12.3-18.4)	18.0 (14.3-22.0)	21.8 (16.6-27.1)	24.6 (18.3-31.3)	27.5 (19.8-35.7)	30.5 (21.4-40.4)	34.6 (23.4-47.1)	37.8 (24.8-52.3)
60-day	11.3 (9.08-13.7)	13.6 (11.0-16.4)	17.3 (14.0-20.9)	20.3 (16.2-24.8)	24.5 (18.7-30.4)	27.6 (20.5-35.0)	30.6 (22.2-39.7)	33.8 (23.8-44.6)	38.1 (25.8-51.5)	41.3 (27.2-56.9)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Reference HEC-HMS (or other hydrologic software) user manuals for guidance on inputting specific parameters into the software program.

4.2.3 Infiltration Losses

Infiltration losses shall be accounted for using the NRCS (formerly SCS) Curve Number method, which is an empirical method developed by the NRCS. The following relationships (Equations 4-4 & 4-5) are used to compute the total runoff for a given total rainfall. In HEC-HMS applications, cumulative totals for rainfall and infiltration are maintained. The total runoff is re-computed for every time step.

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \textbf{Equation 4-4}$$

$$S = \left(\frac{1000}{CN}\right) - 10 \qquad \textbf{Equation 4-5}$$

where: Q = the total runoff (inches);
P = total rainfall in (inches);
S = the amount of rainfall which totally infiltrates before runoff begins (inches); and,
CN= the SCS curve number.

The Curve Number is a function of soil structure, antecedent watershed moisture, and land use. Soil structure is defined by assigning individual soils to one of four hydrologic soil groups (A through D) that represent a wide range of soil porosities. Soils belonging to hydrologic soil group A are the most porous, while soils in group D are the least porous. The hydrologic soil group may be determined from the Soil Survey for Nueces County, Texas or the Soil Survey Geographic Database (SSURGO) for Nueces County. Table 4-4 provides a summary of NRCS Curve Numbers for various land uses.

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Table 4-3. NRCS Curve Numbers

	Hydrologic Soil Group			
Land Use Description	A	B	C	D
Cultivated Land				
Without Conservation Treatment	72	81	88	91
With Conservation Treatment	62	71	78	81
Pasture or Range Land				
Poor Condition	68	79	86	89
Good Condition	39	61	74	80
Meadow: Good Condition	30	58	71	78
Wood or Forest Land				
Thin Stand, Poor Cover, No Mulch	45	66	77	83
Good Cover	25	55	70	77
Open Spaces, Lawns, Parks, Cemeteries				
Good Condition, 75% Grass Cover	39	61	74	80
Poor Condition, 50-75% Grass Cover	49	69	79	84
Commercial and Business Areas (85% Impervious)	89	92	94	95
Industrial Districts (72% Impervious)	81	88	91	93
Residential				
Average Lot Size — *Average % Impervious*				
1/8 acre or less — *65*	77	85	90	92
1/4 acre — *38*	61	75	83	87
1/3 acre — *30*	57	72	81	86
1/2 acre — *25*	54	70	80	85
1 acre — *20*	51	68	79	84
Paved Parking Lots, Roofs, Driveways, Etc.	98	98	98	98
Streets and Roads				
Paved with Curbs and Storm Sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

For watersheds with varying land uses and soil types, composite Curve Numbers shall be computed by determining the Curve Number and drainage area associated with each land use and/or soil category. The composite Curve Number shall then be computed using the following formula:

$$CN_w = \frac{\Sigma(CN_i A_i)}{A_T}$$ **Equation 4-6**

where: CN_w = weighted curve number;
CN_i = curve numbers for various land uses and soil types;
A_i = drainage areas corresponding to values of CN_i (acres); and,
A_T = total drainage area (acres).

DRAFT

For HEC-HMS analyses of drainage areas in Corpus Christi, the NRCS Curve Number shall be selected to account for impervious cover. Hence, a value of zero-percent impervious cover shall be entered in HEC-HMS to avoid “double-counting” impervious cover and over-estimating peak flows. For example, Table 4-4 indicates that the Curve Number for an industrial district with soils categorized as hydrologic soil group C is 91. This Curve Number, which accounts for approximately 72-percent impervious cover on the site, should be entered into HEC-HMS along with a value of zero-percent impervious cover. For the City of Corpus Christi, the NRCS initial loss value computed by HEC-HMS is used and the initial loss parameter is not entered.

4.2.4 Unit Hydrograph Method

Unit hydrographs shall be based on the Clark Unit Hydrograph method, which is one of the unit hydrograph methods available in the HEC-HMS program. The Clark Unit Hydrograph method uses three parameters to define a unit hydrograph for a watershed: the T_c, a storage coefficient, and a time-area curve.

The time of concentration (T_c) is defined as the time required for all portions of the watershed to contribute runoff at the computation point.

The storage coefficient (R) is an indicator of the available storm water storage volume within a watershed within depressions, ponds, channels and floodplains. The value of R varies directly with the relative amount of storage volume within a watershed (i.e., the greater the storage volume, the higher the storage coefficient).

The time-area curve relates the percentage of the watershed contributing runoff at the analysis point to the fraction of the T_c, which has elapsed since the beginning of runoff. The entire watershed is considered to be contributing runoff at the outlet when the elapsed time is equal to or greater than the T_c. This standard curve is applicable as long as extremes in watershed shapes (i.e., very large or very small ratios of watershed length to width) are avoided. Calculation of the time-area curve is handled internally by HEC-HMS with a standard time-area curve based on assumed watershed shape.

In order to calculate T_c and R for drainage areas greater than 200 acres, Equations 4-7 through 4-10 were developed to correlate T_c and R to the hydrologic characteristics of the drainage area. These hydrologic characteristics include the length, slope and roughness of the basin’s longest watercourse, the average watershed slope, and the effective imperviousness of the watershed. T_c and R unit hydrograph parameters are entered into the *transformation* window of the HEC-HMS program *basin model*.

$$T_c + R = 128\frac{\left(\frac{L}{\sqrt{S}}\right)^{0.57} n^{0.8}}{(S_0)^{0.11}(10)^{I}}$$ **Equation 4-7**

$$T_c = (T_c + R) \times 0.38(\log S_0)$$ **Equation 4-8**

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$$R = (T_c + R) - T_c$$ **Equation 4-9**

where: T_c = Clark's time of concentration (hours);
R = Clark's storage coefficient (hours);
L = length of the longest watercourse for the drainage area (miles);
S = average slope along the longest watercourse (ft/mile);
n = Manning's weighted roughness coefficient along the longest watercourse;
S_0 = average basin slope of land draining overland into the longest watercourse (ft/mile); and,
I = effective impervious ratio.

The effective impervious ratio (I) used in Equation 4-7 is determined by:

$$I = CD(10^{-4})$$ **Equation 4-10**

where: C = the average impervious cover of the developed area (percent)
D = percent of the sub-area that is developed

If a HEC-HMS model is developed to perform detention routing (see Chapter 7) or other hydrologic computations for drainage areas less than or equal to 200 acres, then the T_c should be determined using the NRCS Uplands Method discussed in Section 4.1.5 and the R value should be estimated from Equation 4-11:

$$R = 3T_c$$ **Equation 4-11**

where: R = Clark's storage coefficient (hours)
T_c = Clark's time of concentration, calculated from the NRCS Uplands Method (hours)

This relationship between R and T_c is a reasonable assumption for the Gulf Coast area.

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4.2.5 Streamflow Routing

Streamflow routing is the process by which the lagging and attenuating effects of travel time and storage on runoff hydrographs are taken into account as flood flows move from one analysis point to another. Although the HEC-HMS program offers a number of streamflow routing methods, the City of Corpus Christi requires the use of the Modified Puls method where channel cross-sections and a HEC-RAS hydraulic model of the channel are available. For streamflow routing along channels without a HEC-RAS model, the Muskingum-Cunge Standard, Muskingum-Cunge 8-Point, or Muskingum methods should be used depending on which method is best suited to the specific application. However, if backwater conditions and/or overland flooding are anticipated, it is recommended that a HEC-RAS model of the channel be developed and the Modified Puls method be used.

The Modified Puls Method explicitly accounts for the effects of storage volume within the floodplain and is based on a simple continuity equation:

$$\Delta S = I - O \qquad \textbf{Equation 4-12}$$

where: ΔS = change in storage volume within the routing reach;
I = inflow to the routing reach; and,
O = outflow from the routing reach.

For the Modified Puls method, input to the HEC-HMS program consists of a set of flow rates and corresponding storage volumes, which are input in the *basin model* routing reach window. Additionally, the number of sub-reaches and initial flow condition are selected in the same window. The Muskingum method is an approximation of the continuity equation (Equation 4-12) where storage is modeled as the sum of prism and wedge storage. Required input parameters for this method include: the Muskingum K, Muskingum X (ranges from 0.0 to 0.5), and the number of sub-reaches. Refer to, the HEC-RAS & HEC-HMS documentation mentioned in Section 4.2 for additional information on these routing methods.

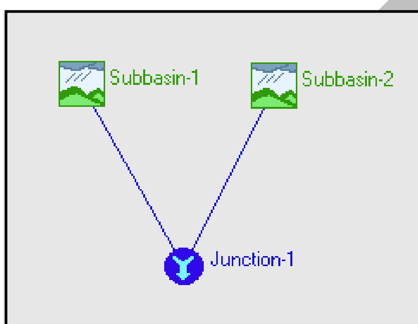
HEC-HMS modeling input for the Muskingum-Cunge Standard method consists of the following physical parameters: the length and slope of the routing reach, the Manning roughness coefficient (n value), the shape of the channel (circular or prismatic), the bottom width or diameter, and the side slope ratio. This mathematical routing method provides an implicit accounting of storage within the channel. However, storage within the floodplain outside the defined channel is not considered. Although the same equations and solution techniques are used for the Muskingum-Cunge 8-Point method, the channel is described with eight station- elevation coordinates instead of a standard cross-section shape. Other required input items for this method are the reach length, energy slope, and n values for the channel and overbanks. For additional information on these routing methods, refer to the HEC-HMS documentation listed in Section 4.2.5.

DRAFT

4.2.6 Combining Hydrographs

When analyzing Basins or Sub-basins that have been divided into two or more sub-areas, it is necessary to combine runoff hydrographs from the individual sub-areas. Combining the hydrographs yields a single hydrograph, which accounts for all the runoff from the individual sub-areas. This is accomplished by inserting a junction in the HEC-HMS Basin Model (or other software models). Connect the two sub-areas to the junction to obtain a combined hydrograph as shown in Figure 4-5. In this Figure, HEC-HMS will compute individual hydrographs for Subbasin-1 and Subbasin-2 and combine them at Junction-1.

Figure 4-1. HEC-HMS Hydrograph Combining Schematic



4.2.7 Green and Ampt Loss

Infiltration losses may also be estimated using the Green and Ampt method. The Green and Ampt method includes

parameters such as hydraulic conductivity (in/hr), wetting font capillary pressure or suction head (in), saturated moisture content (in/in), and initial moisture content (in/in). Resources for additional guidance include “Harris County Flood Control District Hydrology & Hydraulics Manual”, “San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling”, and the “HEC-HMS Technical Reference Memo”. Engineers shall use judgment in applying parameters from those sources as appropriate for Corpus Christi hydrologic conditions.

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Chapter 5
HYDRAULICS

The purpose of this chapter is to provide detailed information on the hydraulic analysis and design of all projects within the City of Corpus Christi. Hydraulic analyses will be required in support of all drainage studies and construction drawings involving Minor, Intermediate, and Major drainage facilities, plats, and permits.

5.1 LEVEL OF PROTECTION FOR STORM WATER INFRASTRUCTURE

Reference section 2.7 for Level of Protection (Design Recurrence Interval) and freeboard requirements for storm water infrastructure.

5.2 GENERAL DESIGN REQUIREMENTS FOR OPEN CHANNELS

5.2.1 Hydraulic Design

5.2.1.1 For studies involving Federal Emergency Management Agency (FEMA) submittals, the 10-, 50-, 100-, and 500-year design storm events must be analyzed regardless of the classification of the open channel system.

5.2.1.2 Storm water infrastructure within TxDOT ROW must be designed in accordance with TxDOT requirements.

5.2.1.3 Storm water infrastructure draining to TxDOT ROW must be designed so that downstream TxDOT infrastructure is not adversely impacted.

5.2.1.4 Hydraulic design shall be performed using industry standard software programs.

5.2.1.5 For storm water infrastructure within or impacting the FEMA floodplain, use software programs approved by FEMA.

5.2.2 Design Requirements for Earthen Channels

a. Rural Roadside Ditches

i. Side Slopes 3:1(H:V) or flatter
ii. Freeboard not required
iii. Minimum Velocity is 2 fps when flowing full
iv. Erosion protection
   1. Vegetative cover shall be established in all ditches
   2. Erosion protection mats and topsoil may be required to protect against erosion and to help establish vegetation
v. Drainage easements shall be required if the ditch cannot be fully contained within the road ROW

DRAFT

b. Earthen Channels
   i. Includes temporary ditches
   ii. Bottom width shall be 6 feet minimum
   iii. Side sloes shall be 4:1 (H:V) or flatter
   iv. Freeboard shall be as shown in Section 2.7
   v. Maintenance strip of 20 feet minimum width shall be required on both sides of the channel
   vi. Maximum velocity allowed shall be 5 fps unless erosion-resistant treatments are included
   vii. Erosion protection
      1. Vegetative cover shall be established in all ditches
      2. Erosion protection mats and topsoil may be required to protect against erosion and to help establish vegetation
      3. Stream bed and slope protection required at bends (Radius < 3 times bottom width), confluences (>15°), and outfalls
   viii. Avoid runoff entering channels over the banks by means of inlets and/or backslope swales. See Figure 5-1 below.
   ix. Earthen channels may be designed to accommodate existing, interim, or ultimate levels of development within the watershed, depending on the stage of development. However, right-of-way (ROW) widths and drainage easements shall be based on the required channel dimensions to accommodate peak discharge rates from a fully- developed watershed at ultimate conditions.
   x. Channels and channel ROWs must be at least 85% vegetated promptly within 60 days after construction.
   xi. For utilities located within the drainage easement, the minimum setback from the top of the slope is based upon the depth of the utility and a one-to-one horizontal to vertical (1:1 H:V) slope from the channel top of bank to the invert of the pipe. Additionally, for each 10 feet of depth, a 5-foot minimum horizontal bench must be added.

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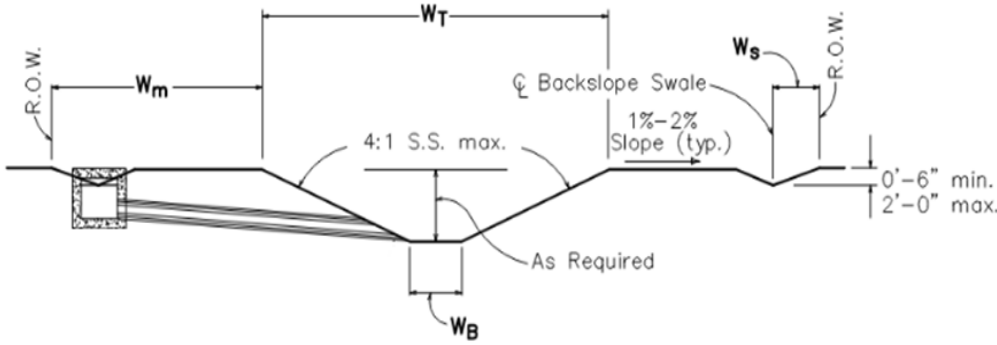
Figure 5-1. Typical Cross-Section Trapezoidal Earthen Channel



W_T = Top width of channel

W_m = Width of maintenance strip
Minimum W_m = 20ft

W_s = Distance from centerline of backslope drainage swale to the channel R.O.W. line
Minimum W_s = 5ft

W_B = Bottom width of channel
Minimum W_B = 6ft

5.2.3 Design Requirements for Concrete-Lined Channels

a. Concrete-lined channels shall be designed to meet the following minimum requirements (See Figure 5-2).

i. Bottom width shall be 8 feet minimum
ii. Side slopes shall be 1.5:1 (H:V) or flatter
iii. Side slopes steeper than 1.5:1, and vertical channel walls shall include structural engineering design.
iv. Freeboard shall be 1 foot minimum for design year event
v. Maintenance strip of 20 feet minimum shall be required on both sides of the channel
vi. Maintenance Access shall be required at crossing structures and at 2000-ft intervals
vii. Maximum velocity allowed shall be 12 fps.
viii. A geotechnical investigation and report on local soil conditions is required for all concrete channel construction and improvement projects.
ix. All concrete slope paving shall consist of Class A concrete, minimum.
x. Concrete channel bottoms shall be Class C concrete, minimum.
xi. Where construction is to take place under muddy conditions or where standing water is present, a slab of Class C concrete shall be placed in the channel bottom prior to placement of the concrete slope paving.
xii. Provide minimum toe-downs as follows: 18” toe-downs at the top of side slopes; 24” toe-downs at the downstream ends of channels in clay soils; and 36” toe-downs at the downstream end of channels in sandy soils; and 36” toe-downs at the upstream ends of all concrete channels.
xiii. Channels ROWs must be vegetated immediately after construction. 85% re-vegetation coverage is required within 60 days after construction.
xiv. Weep holes shall be used to relieve hydrostatic pressure behind lined channel sections. The specific type, size, and placement of the weep holes shall be based on the recommendations of the geotechnical report.
xv. Control joints shall be provided at a maximum spacing of 25 feet. A sealing agent shall be utilized to prevent moisture infiltration at control joints.
xvi. Concrete slope protection shall have the minimum thickness and reinforcement specified in
xvii. For utilities located within the drainage easement, the minimum setback from the top of the slope is based upon the depth of the utility and a 1 horizontal to one vertical (1:1) slope from the channel top of bank to the invert of the pipe. Additionally, for each 10- feet of depth, a 5-foot minimum horizontal bench must be added.

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Figure 5-2. Typical Cross-Section Trapezoidal Concrete-Lined Channel

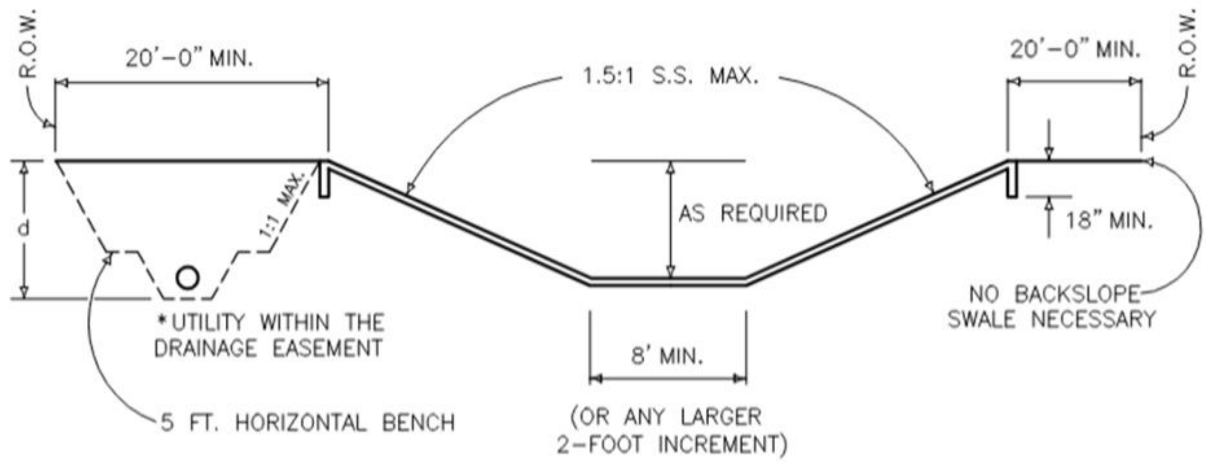

Table 5-1. Minimum Thickness and Reinforcement for Concrete Channel Lining

Channel Side Slope (H:V)	Minimum Thickness (inches)	Minimum Reinforcement	
		Material	Dimensions
3:1	4	steel rebar	To be determined by Registered Engineer
2:1	5	steel rebar	To be determined by Registered Engineer
1.5:1	6	steel rebar	To be determined by Registered Engineer

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5.2.4 Design Requirements for Concrete Low-Flow Sections

Concrete low-flow sections can be incorporated into designs for earthen and concrete-lined channels to provide additional capacity or depth in areas where channel ROW is limited.

a. The following criteria shall be used for concrete low-flow sections:

   i. A geotechnical investigation and report on local soil conditions is required for all channel construction and improvement projects.
   ii. All concrete slope paving shall consist of Class A concrete.
   iii. The reinforcing steel design shall be based on the use of ASTM A-615, Grade 60 steel.
   iv. The minimum bottom width of the low-flow section shall be six feet.
   v. For bottom widths of 12 feet or more, the channel bottom shall be graded toward the centerline at a slope of at least 2%.
   vi. Where construction is to take place under muddy conditions or where standing water is present, a slab of Class C concrete shall be placed in the channel bottom prior to placement of the concrete slope paving.
   vii. Control joints shall be provided at a maximum spacing of 25 feet. A sealing agent shall be utilized to prevent moisture infiltration at control joints.

5.2.5 Design Requirements for Transitions, Bends, and Confluences

a. Transitions, bends, and confluences shall be designed to meet the following minimum requirements:

   i. Transitions in channel bottom widths or side slopes shall be designed to create minimal flow disturbance and energy loss. Transition angles shall be less than 12 degrees.
   ii. A warped or wedge-type transition is recommended for connecting rectangular and trapezoidal channel sections.
   iii. Channel bends shall be made as gradually as possible. The minimum bend radius along the centerline of the channel is three times the bottom width of the channel. Where smaller radii are required, erosion protection (i.e., concrete slope paving, rock riprap, interlocking blocks, etc.) is required as specified in Chapter 10.
   iv. The maximum allowable deflection angle for channel bends is 90 degrees.
   v. Erosion protection shall be provided at channel confluences in accordance with the erosion protection requirements described in Chapter 10. See Figure 5-3 below.

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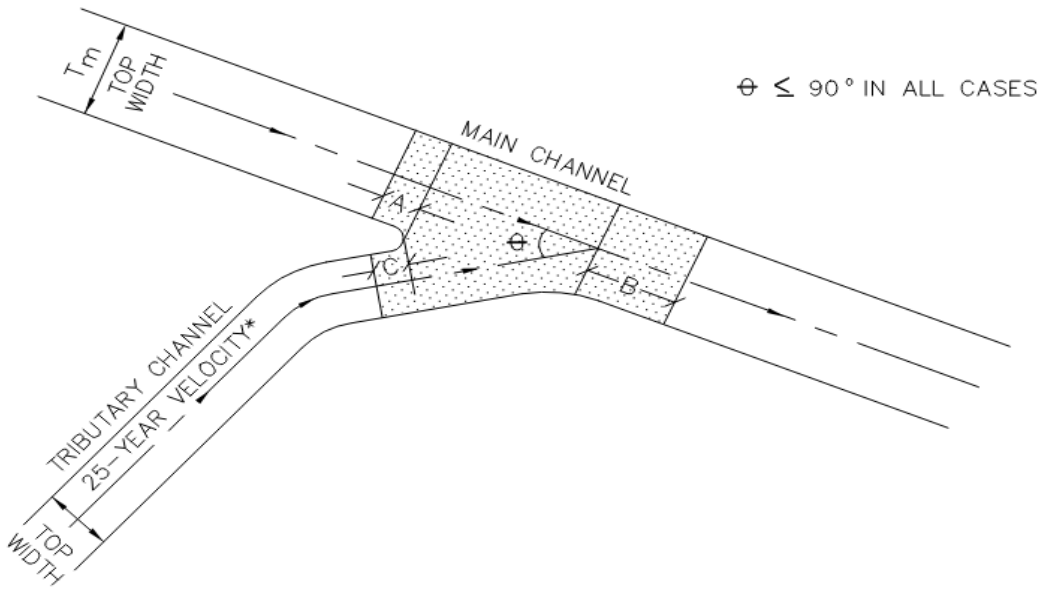
Figure 5-3. Erosion Protection at Stream Confluences



MINIMUM EROSION PROTECTION		
VELOCITY* IN TRIBUTARY CHANNEL (FEET PER SECOND)	ANGLE OF INTERSECTION, θ	
	15° – 45°	45° – 90°
4 OR MORE	PROTECTION	PROTECTION
2 TO 4	NO PROTECTION	PROTECTION
2 OR LESS	NO PROTECTION	NO PROTECTION

MINIMUM EXTENT OF EROSION PROTECTION	
LOCATION	MINIMUM DISTANCE (feet)
A	20
B	LARGER OF 50 OR 0.75 T_m/TAN θ
C	20

* NOTE:

VELOCITY IN TRIBUTARY CHANNEL ASSUMING NO BACKWATER FROM MAIN CHANNEL.

5.2.6 Design Requirements for Culverts

a. Culverts shall be designed to meet the following requirements:

   i. Minimum size per IDM 3.04b.
   ii. Freeboard not required
   iii. Maximum velocity is 15 fps
   iv. Design loading is HS20-44 highway loading
   v. Bedding for RCP shall be granular bedding as shown on Table 1 of City Storm Water Standard Details
   vi. Bedding for RCB shall be cement stabilized sand bedding
   vii. Backfill per City Standard Specifications 022020 and 027402
   viii. Stream bed and slope protection required at upstream and downstream ends of culverts
   ix. Culvert ends shall be protected from traffic impacts by the use of Safety End Treatments (SETs)
   x. Culverts 24-increas and larger will be protected from traffic impacts by SETs with steel cross-pipes
   xi. Where possible, culverts should be aligned parallel to the longitudinal axis of the channel. At locations where a skewed angle is required, the change in alignment should be accomplished upstream of the culvert so that the culvert is aligned with the downstream channel.
   xii. Concrete rip rap or stone rip rap shall be used upstream and downstream of the culvert to protect earthen channels from erosion.
   xiii. Culverts shall extend across road and ROWs at crossing locations to allow for future pavement widths according to street classification.
   xiv. Hydraulic jumps must be contained within the limits of the channel, and within an area protected by concrete rip rap and/or stone rip rap.
   xv. All pre-cast reinforced concrete pipe shall be ASTM C-76.
   xvi. All pre-cast reinforced concrete box culverts with more than two feet of earthen cover shall be ASTM C789-79. All pre-cast reinforced box culverts with less than two feet of earthen cover shall be ASTM 850-79.
   xvii. Joint sealing materials for pre-cast concrete culverts shall comply with the "ASHTO Designation M-198 74 I, Type B, Flexible Plastic Gasket (Bitumen)" specification.
   xviii. Two-sack-per-ton cement-stabilized sand shall be used for backfill around culverts.
   xix. A six-inch bedding of two-sack-per-ton cement-stabilized sand is required for all pre-cast concrete box culverts.

5.2.7 Design Requirements for Bridges

a. Bridges shall be designed to meet the following requirements:

   i. Freeboard is 1 foot minimum from low chord to design-year event WSE
   ii. Maximum velocity is 8 fps
   iii. Design loading is HS20-44 highway loading

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iv. Bridge design submittals must include hydrologic and hydraulic analysis, structural design plans and a geotechnical engineering report.
v. Bridge Stream bed and slope protection may be required based on structural, slope stability, and geotechnical considerations.
vi. New bridges should be designed to completely span the existing or proposed channel so that the channel will pass between the abutments without significant contractions or changes in the channel shape.
vii. Bridges constructed on existing or interim channels shall be designed to accommodate the ultimate channel section with minimum structural modifications.
viii. Bridges should intersect a channel at an angle of 90 degrees, if possible.
ix. Pier bents and abutments shall be aligned parallel to the direction of flow in the channel. Pier bents shall be placed as far from the center of the channel as possible and wherever possible shall be placed within the channel side slopes instead of the channel bottom.
x. Where hydraulic jumps are anticipated around bridges, the channel geometry shall be modified to force the hydraulic jump to occur in a portion of the channel protected with concrete slope paving.
xi. Refer to the TxDOT Hydraulic Design Manual for further information related to designing and analyzing bridges.

5.2.8 Maximum Allowable Velocities

The maximum allowable velocity in open channels and at bridges or culverts shall be analyzed for the associated design storm event. When designing in areas with sandy soils or evidence of erosion, use of velocities lower than the maximum allowable is recommended.

5.3 HYDRAULIC ANALYSIS OF OPEN CHANNELS

This section describes the methods to be used in the hydraulic analysis of open channels as well as associated bridge and culvert structures.

5.3.1 Acceptable Channel, Culvert, and Bridge Methods

Channel capacity and dimensions can be determined using normal depth channel calculations in situations where a channel has a uniform geometric cross section, the channel depth and WSE is not controlled by backwater, and the channel is not in the FEMA floodplain.

All other channel capacity and dimension calculations shall be performed using HEC-RAS or other FEMA approved software.

Culvert capacity calculations for single box culverts (SBCs) or multiple box culverts (MBCs) with an opening width less than 20’ may be performed with software such as HY-8 or other culvert calculation software. For culverts with a width of 20’ or more, and for any culvert within the FEMA floodplain, use HEC-RAS or other FEMA approved software.

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Bridge hydraulic calculations shall be performed using HEC-RAS or other FEMA approved software.

5.3.2 Flow Data

The engineer is responsible for calculating design flow data for channels, culverts, and bridges, as described in Chapter 4, Hydrology.

5.3.3 Boundary Conditions

In order to perform channel computations in software like HEC-RAS, boundary conditions or starting WSELs must be defined. Boundary conditions are required at the downstream and upstream ends of the river system depending on the flow regime used. Refer to the guidance manual of the software used to input parameters and perform calculations. Subcritical flow typically occurs in the Corpus Christi area. This flow regime has a low velocity and appears tranquil, whereas the supercritical flow regime is characterized by more rapid flows.

If an existing hydraulic model of the receiving channel is not available, then normal depth should be used as the downstream boundary condition and the energy slope should be approximated as the slope of the bottom of the channel. If a model of the receiving channel is available and the tailwater in this channel can be determined, then the known WSEL downstream boundary condition should be selected.

5.3.4 Cross-Section Data

Cross-sections for flood studies and channel designs should be spaced approximately every 500 feet along a channel, and where channel dimensions, slopes, or tailwater conditions change. Channel topographic data should be based on survey for design purposes, and based on LIDAR at minimum for non-design analysis purposes. Survey and/or bathymetry may be required based on the presence of base flow or ponding within a channel system.

Contraction and expansion coefficients are required for 1D floodplain studies. For the Corpus Christi area, the typical expansion and contraction coefficients for open channels are 0.1 and 0.3, respectively. Higher coefficients of 0.3 and 0.5 should be used at bridges and culverts to simulate expansion and contraction conditions around these structures. Reference the modeling guidelines for the appropriate software for information relating to inserting and modifying cross-sectional data.

5.3.5 Manning's Roughness Coefficients

Manning's Equation is used to compute conveyance and flows in open channels. The n value used in this method varies inversely with conveyance and is a measure of the roughness of the surfaces with which storm water comes into contact. For example, a forested area would have a higher n value and a lower conveyance than a pasture or open field. Recent aerial photographs as well as field reconnaissance can be used in

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conjunction with Table 5-2 to determine *n* values for channels, floodplains, and overbank areas.

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Table 5-2. Manning's Roughness Coefficients

Type of Channel and Description	Roughness Coefficient (n) Minimum	Normal	Maximum
Reinforced Concrete Pipe/Reinforced Concrete Box	0.012	0.013	0.017
Excavated or Dredged Channels			
Concrete Lined Channels	0.011	0.013	0.015
Earthen Channels, Straight and Uniform			
Clean, After Weathering	0.016	0.018	0.020
With Short Grass, Few Weeds	0.022	0.027	0.033
Earthen Channels, Winding and Sluggish			
No Vegetation	0.023	0.025	0.030
Grass, Some Weeds	0.025	0.030	0.033
Dense Weeds or Plants in Deep Channels	0.030	0.035	0.040
Earth Bottom and Rubble Sides	0.028	0.030	0.035
Stony Bottom and Weedy Banks	0.025	0.035	0.040
Cobble Bottom and Clean Sides	0.030	0.040	0.050
Channel Not Maintained, Weeds & Brush Uncut			
Dense Weeds, High as Flow Depth	0.050	0.080	0.120
Clean Bottom, Brush on Sides	0.040	0.050	0.080
Same, Highest Stage of Flow	0.045	0.070	0.110
Dense Brush, High Stage	0.080	0.100	0.140
Natural Streams			
Clean, Straight, Full Stage, No Rifts or Deep Pools	0.025	0.030	0.033
Same as Above, But Some Stones and Weeds	0.030	0.035	0.040
Clean, Winding, Some Pools and Shoals	0.033	0.040	0.045
Same as Above, But Some Weeds and Stones	0.035	0.045	0.050
Same as Above, Lower Stages, More Ineffective Areas	0.040	0.048	0.055
Sluggish Reaches, Weedy, Deep Pools	0.050	0.070	0.080
Floodplains/Overbanks			
Pasture, No Brush			
Short Grass	0.025	0.030	0.035
High Grass	0.030	0.035	0.050
Cultivated Areas			
No Crop	0.020	0.030	0.040
Mature Row Crops	0.025	0.035	0.045
Mature Field Crops	0.030	0.040	0.050
Brush			
Scattered Brush, Heavy Weeds	0.035	0.050	0.070
Light Brush and Trees, in Winter	0.035	0.050	0.060
Light Brush and Trees, in Summer	0.040	0.060	0.080
Medium to Dense Brush, in Winter	0.045	0.070	0.110
Medium to Dense Brush, in Summer	0.070	0.100	0.160
Trees			
Dense Willows, Summer, Straight Channel	0.110	0.150	0.200
Cleared Land with Stumps, No Brush	0.030	0.040	0.050
Cleared Landwith Brush	0.050	0.060	0.080
Heavy Stand of Timber, Flood Stage Below Branches	0.080	0.100	0.120
Heavy Stand of Timber, Flood Stabe in the Branches	0.100	0.120	0.160

Note: Use coefficients from "Normal" column for design. Coefficients from "Minimum" and "Maximum" columns may be used for analyzing existing systems.

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Although Table 5-2 is generally adequate for selecting *n* values corresponding to existing field conditions, project-specific considerations may warrant the use of Equation 5-1 for a more detailed determination of *n* values associated with the channel and floodplains (overbanks). For most applications, it is acceptable to round *n* values to the nearest 0.001.

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m$$ **Equation 5-1**

where: n = composite Manning's roughness coefficient;
n_0 = base value for the bare soil surface material of the channel or floodplain; n_1 = value to correct for the irregularity of the channel or floodplain;
n_2 = value to account for variations in the shape and size of the channel or floodplain cross-section;
n_3 = value to account for obstructions in the channel or floodplain; n_4 = value to account for the effects of vegetation; and,
m = correction factor for the sinuosity of the channel or floodplain.

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Tables 5-3 and 5-4 provide a summary of parameters used in Equation 5-1 to compute n values for channels and floodplains, respectively.

Table 5-3. Parameters Used in Computing Manning's Roughness for Channels

Parameter	Accounts For	Representative Roughness Values
n_0	Channel Material	0.011 for Concrete 0.020 for Earth 0.025 for Rock Cut 0.024 for Fine Gravel 0.028 for Coarse Gravel
n_1	Degree of Irregularity	0.000 for Smooth 0.005 for Minor Irregularities 0.010 for Moderate Irregularities 0.020 for Severe Irregularities
n_2	Variation of Channel Cross-Section	0.000 for Gradual Variations 0.005 for Alternating Occasionally 0.010 to 0.015 for Alternating Frequently
n_3	Relative Effect of Obstructions	0.000 for Negligible Obstructions 0.010 to 0.015 for Minor Obstructions 0.020 to 0.030 for Appreciable Obstructions 0.040 to 0.060 for Severe Obstructions
n_4	Vegetation	0.005 to 0.010 for Low Vegetation 0.010 to 0.025 for Medium Vegetation 0.025 to 0.050 for High Vegetation 0.050 to 0.100 for Very High Vegetation
m	Degree of Meandering	1.000 for Minor Meandering 1.150 for Appreciable Meandering 1.300 for Severe Meandering

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Table 5-4. Parameters Used in Computing Manning's Roughness for Floodplains/Overbanks

Parameter	Accounts For	Representative Roughness Values
n_0	Base Material	0.010 for Concrete 0.020 for Earth 0.025 for Rock Cut 0.024 for Fine Gravel 0.028 for Coarse Gravel
n_1	Degree of Irregularity	0.00 for Smooth 0.01 to 0.005 for Minor Irregularities 0.006 to 0.010 for Moderate Irregularities 0.011 to 0.020 for Severe Irregularities
n_2	Variation in Cross-Section	0.000 Not Applicable
n_3	Effect of Obstructions	0.000 to 0.004 for Negligible Obstructions 0.005 to 0.019 for Minor Obstructions 0.020 to 0.030 for Appreciable Obstructions
n_4	Amount of Vegetation	0.001 to 0.010 for Small Amounts 0.011 to 0.025 for Medium Amounts 0.026 to 0.050 for Large Amounts 0.051 to 0.100 for Very Large Amounts 0.100 to 0.200 for Extreme Amounts
m	Degree of Meander	1.00 Not Applicable

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5.3.6 Bridge & Culvert Data

Data pertaining to the following is typically required for bridge and culvert data design.

- Deck/roadway data;
- Geometric data for culverts or bridges;
- Sloping abutments and pier data for bridges; and,
- The bridge or culvert modeling approach.
- The width of the bridge deck/culvert;
- Upstream and downstream bridge deck/culvert station-elevation data;
- Overflow weir coefficient, which ranges from 2.6 for flow over a bridge deck to 3.0 for flow over elevated roadway approach embankments;
- Upstream and downstream embankment side slopes;
- The minimum weir flow elevation; and the weir crest shape.
- Roughness coefficients and expansion/contraction coefficients as shown in the tables below.
- Field/survey data should be used for analyzing existing and designing proposed culvert and bridge structures.

Refer to modeling guidelines for the appropriate bridge and culvert software. Refer to the TxDOT Hydraulic Design Manual for further information related to bridge and culvert modeling.

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Table 5-5. Manning's Roughness Coefficients for Culverts

Description of Pipe	Roughness Coefficient (*n*)
Reinforced Concrete Pipe (RCP) and Box Culverts	0.013
HDPE Plastic Pipe	0.012
Corrugated Metal Pipe (CMP) with 2-2/3" x 1/2" Corrugations	0.024
Corrugated Metal Pipe (CMP) with 3" x 1" Corrugations	0.027
Corrugated Metal Pipe (CMP) with 6" x 2" Corrugations	0.030

Table 5-6. Entrance Loss Coefficients for Culverts

Type of Structure and Configuration of Entrance	Coefficient (K_e)
Concrete Pipe Culverts	
Projecting from Fill	
Socket End (Groove End) of Pipe	0.2
Square-Cut End of Pipe	0.5
Headwall or Headwall & Wingwalls	
Socket End of Pipe (Groove End)	0.2
Square Edge	0.5
Mitered to Conform to Fill Slope	0.7
End Section Conforming to Fill Slope	0.5
Corrugated Steel Culverts	
Projecting From Fill	0.9
Headwall or Headwall & Wingwalls	0.5
Mitered to Conform to Fill Slope	0.2
End Section Conforming to Fill Slope	0.5
Concrete Box Culverts	
Headwall Parallel to Embankment (No Wingwalls)	0.5
Wingwalls at 30 Degrees to 75 Degrees to Barrel	0.4
Wingwalls at 10 Degrees to 25 Degrees to Barrel	0.5
Wingwalls Parallel (Extensions of Sides)	0.7

DRAFT

5.3.7 Floodway Analysis

A floodway analysis is required if modifications are made to flood study models or proposed physical changes are proposed that would impact FEMA floodways. As described in Chapter 2 and illustrated on Figure 2-1, the floodway is a corridor of effective flow that includes the channel and any adjacent land areas required to pass the 100-year peak discharge rates without increasing the WSE at any point along the channel more than one-foot above the 100-year base flood elevations (BFE). The floodway is a regulatory concept that is intended to prevent encroachments (i.e., fill, structures, or other obstructions) from being placed too close to the channel without sufficient analysis and mitigation measures. Additional information on FEMA requirements for floodway analysis can be obtained from FEMA's website.

5.4 GENERAL DESIGN REQUIREMENTS FOR UNDERGROUND STORM SEWERS

This section describes the general design requirements for underground storm sewers, which include all closed conduit systems and may consist of either storm sewer pipes or box culverts.

5.4.1 Design Storm Frequencies

Reference section 2.7.2 for design storm frequencies. Infrastructure within TxDOT ROW must be designed according to TxDOT requirements.

5.4.2 General Design Requirements

Hydraulic analysis of underground (enclosed) storm sewers may be performed using manual

a. Underground storm sewers shall be designed to meet the following requirements:

   i. Hydraulic design shall be performed using software tools commonly used for storm sewer design and approved by the City.
   ii. All hydraulic analyses shall include the appropriate tailwater in the calculations.
      1. Tailwater elevations shall be determined through appropriate engineering analysis, known water surface elevations for the flood interval, FEMA FIRMs, City storm water models, or other resources deemed acceptable by the City Engineer
   iii. Pipe material shall be reinforced concrete pipe (RCP) or box (RCB) under roadway pavement.
   iv. Minimum stormwater pipe size is 18" regardless of location
   v. Minimum RCP size under pavement is 18" for lateral lines or trunk lines
   vi. Minimum RCB size is 3' x 2'
   vii. Minimum pipe slopes shall be per Table 5-7 below.
   viii. Minimum Cover
      1. 2' for Class III RCP as defined by ASTM C-76
      2. 1' for Class IV RCP as defined by ASTM C-76

- ix. Pipe Velocity
  1. 2 fps minimum (flowing full) for a 5-year storm where obtainable
  2. Maximum 15 fps in trunk lines
  3. No maximum velocity for laterals
- x. Manholes (MH)
  1. 600’ maximum spacing
  2. MR required at each change in pipe size
  3. MH required at directional changes/deflections > 5°
  4. MH alternatives include junction box or post/curb inlets
  5. Rises may be used on box culverts except where box size changes
- xi. Hydraulic Grade Line (HGL)
  1. At or below rim elevation at MH rims for minor design-year event per UTP designation
  2. At or below top of curb for major design-year event per UTP designation
- xii. Downstream pipe soffit elevation should be at or below the upstream pipe soffit elevation

5.4.3 Extreme Event Design

The capacity of the storm sewer system might be exceeded during rainfall events that are more intense than the design storm. For example, a Minor storm sewer system designed to convey the five-year peak runoff rates from a drainage area will not have adequate capacity to convey the 100-year peak runoff rates from that area. Ponding may occur in streets, roadside ditches, and adjacent low-lying areas when the capacity of the storm sewer system is exceeded. To eliminate or reduce potential flooding on adjacent properties, street layout and pavement grades shall be designed to direct storm water runoff into channels or drainage systems without flowing through private property. The street grading plan shall be developed to ensure 100-year ponding levels in the streets remain below habitable living space.

Special care should be taken when grading cul-de-sac, knuckle, T-intersections, and other intersection configurations, so that runoff does not leave the street ROW and flood properties and buildings.

In areas where streets cannot be graded to carry sheet flows directly to an open channel, an extreme event overflow structure must be provided to collect sheet flow and convey it to a channel. This structure may be an oversized storm sewer inlet or a grass- or concrete-lined channel located within a drainage easement between two residential lots. In either case, the extreme event overflow structure and associated drainage structures must be designed to convey the 100-year peak runoff rate from the developed drainage area.

5.5 HYDRAULIC ANALYSIS OF UNDERGROUND STORM SEWERS

This section describes the requirements for the hydraulic analysis of underground storm sewer systems.

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5.5.1 Acceptable Storm Sewer Design Methods

Hydraulic analysis of underground (enclosed) storm sewers may be performed using manual calculations, spreadsheet, or other acceptable calculation software. approved by the City. The hydraulic analysis and design procedure for storm sewers using manual or spreadsheet calculations is described in Section 5.5.4**.**

5.5.2 Peak Runoff Rates

Peak runoff rates shall be calculated as described in Chapter 4, Hydrology. Design storm events are described in section 2.7.2.

5.5.3 Storm Sewer Slopes

As indicated in Section 5.4.2, the minimum allowable velocity for storm sewers flowing full is two feet per second and the maximum allowable velocity for storm sewers flowing full is fifteen feet per second. Manning's Equation (Equation 3-1) can be rearranged by incorporating the Continuity Equation (Q = VA), to solve for velocity.

Incorporating the Continuity Equation into Manning's Equation yields Equation 5-2:

$$V = \left(\frac{1.49}{n}\right) R^{2/3} S^{1/2}$$ **Equation 5-2**

The hydraulic radius can be expressed as a function of the diameter for circular pipes flowing full:

$$R = \frac{D}{4}$$ **Equation 5-3**

where: D = pipe diameter (feet).

Substituting Equation 5-3 into Equation 5-2 and rearranging to solve for the slope of the pipe yields Equation 5-4.

$$S = \left[\frac{V_n}{1.49\left(\frac{D}{4}\right)^{2/3}}\right]^2$$ **Equation 5-4**

DRAFT

A list of minimum slopes to achieve acceptable flow velocities in RCP is provided in Table 5-7. Similar information can be calculated for other pipe materials.

Table 5-7. Minimum Pipe Slope (RCP)

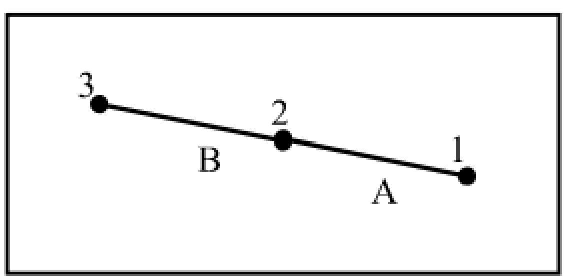
Nominal Pipe Size	Minimum Slope	
(Inches)	(Foot/Foot)	(%)
18	0.00120	0.120%
24	0.00080	0.080%
30	0.00058	0.058%
36	0.00046	0.046%
42	0.00037	0.037%
48	0.00030	0.030%
54	0.00020	0.020%
60	0.00020	0.020%
66	0.00020	0.020%
72	0.00020	0.020%

5.5.4 Hydraulic Analysis and Design Procedure for Storm Sewer Systems

The inlet location shall be dictated on the basis of physical demands and hydraulic

The hydraulic analysis and design procedure for storm sewer systems includes the following items: determining preliminary storm sewer sizes, calculating friction losses, estimating Minor losses, and computing hydraulic grade lines. As shown in Figure 5-4, storm sewer system schematics are useful tools for the analysis and design of storm sewer systems. Manholes and inlets are defined as nodes (i.e., 1, 2, and 3) and storm sewer pipes are defined as links (i.e., A and B) in these schematics.

Figure 5-4.: Storm Sewer System Schematic



5.5.4.1 Preliminary Storm Sewer Sizes

Although final storm sewer sizes are dependent upon the results of the HGL computations described in Section 5.5.4.3, preliminary storm sewer sizes can be

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determined by selecting pipes that carry design peak runoff rates at full friction flow capacity.

Once a system schematic is prepared, the peak runoff rate (Q) at each extreme node (inlet) should be calculated using the appropriate hydrologic method.

Equations 3-1 and 5-3 have been combined and the area (A) has been expressed in terms of the pipe diameter (D) to yield Equation 5-5 for circular pipes flowing full, which can be used to estimate a preliminary size for each storm sewer segment. It is important to note that the preliminary storm sewer sizing is an iterative process, since the minimum and maximum allowable pipe slopes will change with each assumed pipe diameter. If the calculated diameter does not correspond to a commercially available pipe size, then the closest commercially available pipe size that exceeds the required capacity shall be selected. For example, if calculations indicated that a 32-inch diameter pipe is required to convey the Q from the drainage area, then a 36-inch pipe would be selected.

$$D = 1.333\left(\frac{1.49}{S^{1/2}}\right)^{3/8}$$ **Equation 5-5**

Appropriate pipe slopes can be selected from Table 5-9 or calculated using Equation 5-5 and used to assign upstream and downstream flowline (invert) elevations at each node (i.e., manhole or inlet). These elevations should be adjusted as needed to maintain adequate cover on the storm sewer and to achieve the desired system depth.

The design capacity and velocity associated with that capacity should be calculated with Equations 3-1 and 5-2, respectively. The actual velocity of flow through each storm sewer segment can be estimated using the continuity equation (Q=VA) and used to calculate the travel time (T) for flow in each storm sewer segment with Equation 5-6.

$$T = \frac{L}{V}$$ **Equation 5-6**

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Equation 5-7 should be used to determine the T_c at the downstream node of each storm sewer segment (T_{cd}), which corresponds to the upstream node of the next storm sewer segment downstream.

$$T_{cd} = T_{cu} + T$$ **Equation 5-7**

where: T_{cd} = T_c at downstream end (node) of storm sewer segment being analyzed (minutes);

T_{cu} = T_c at upstream end (node) of storm sewer segment being analyzed (minutes).

The Q should be calculated at the upstream end (node) of each storm sewer segment based on the total drainage area contributing to that node, the composite runoff coefficient at that node, and the longest T_c from all of the drainage areas contributing to that node. For example, the longest T_c at Node 2 in Figure 5-23 would be the largest of the following:

a. The T_c for the drainage area contributing at Node 3 plus the T_c through storm sewer segment B; or,
b. The T_c for the drainage area entering the system at Node 2.

It is important to note that Q values from multiple storm sewer segments contributing to a single node cannot be added to compute the Q at that node. In some instances, calculated Q values may decrease as the analysis proceeds downstream. However, the previous Q should be used to avoid designing for a reduced Q. The preliminary storm sewer segment sizing should proceed in a downstream direction and this process should be repeated until a preliminary size has been estimated for all storm sewer segments in the system.

5.5.4.2 Friction Losses

The friction loss (h_f) for each segment of the storm sewer system, which is used in the HGL computations described in Section 5.4.4.4, can be computed with Equations 5-8 through 5-10.

$$h_f = S_f L$$ **Equation 5-8**

where: h_f = friction loss (feet);
L = length of pipe (feet); and,
S_f = friction slope (feet/foot).

As shown in Equation 5-9, Manning's Equation can be rearranged to solve for the friction slope (S_f):

$$S_f = \frac{Q^2 n^2}{(1.49)^2 A^2 R^{4/3}}$$ **Equation 5-9**

Incorporating Equation 5-9 into Equation 5-8 yields Equation 5-10, which can be used to calculate hf.

$$h_f = \frac{(n^2 Q^2 L)}{(2.22 A^2 R^{4/3})}$$ **Equation 5-10**

Substituting Equation 5-3 into Equation 5-10 and expressing the area (A) in terms of the pipe diameter (D) yields Equation 5-11 for circular pipes flowing full.

$$h_f = \frac{4.637 n^2 Q^2 L}{(D^{16/3})}$$ **Equation 5-11**

5.5.4.3 Minor Losses

Minor losses are those losses that result from changes in velocity or direction of flow. Although minor losses in storm sewer systems are usually insignificant, they may exceed the h_f in relatively short storm sewer segments. For this reason, minor losses must be accounted for. In addition, the cumulative effect of minor losses may be significant in relatively flat areas like Corpus Christi. Minor losses include those associated with: pipe entrances, pipe exits, pipe bends, pipe elbows, junctions, manholes, expansions, contractions, and appurtenances such as valves and meters. It is important to note that minor losses can be minimized by careful design. For example, severe pipe bends can be replaced by gradual curves if sufficient ROW is available and costs are manageable. Furthermore, well- designed manholes and inlets without sharp or sudden transitions or flow impediments do not cause significant minor losses.

Minor losses are typically computed using a loss coefficient and flow velocities in upstream and downstream pipe segments. Although entrance losses, exit losses, and losses at inlets and manholes are discussed in detail in this section, it may be necessary to account for other minor losses depending on project specific considerations. For additional information on minor losses refer to TxDOT's *Hydraulic Design Manual* or other hydraulic reference manuals.

Equation 5-12 can be used to compute entrance losses:

$$H_E = K_e \frac{V^2}{2g}$$ **Equation 5-12**

DRAFT

where: H_E = entrance loss (feet);
K_e = the entrance loss coefficient, from Table 5-6, or 1.25 for inlets or manholes at the beginning of a storm sewer segment;
V = velocity in the pipe (feet per second); and,
g = the acceleration of gravity, 32.2 feet/second^2.

For this calculation, the velocity upstream of the pipe entrance is assumed to be zero. Table 5-6 provides a summary of entrance loss coefficients for a number of culvert entrance configurations. Equation 5-12 can also be used to compute exit losses and the exit loss coefficient may be assumed to be equal to 1.0 for most applications.

Minor losses at inlets and manholes can be computed with Equation 5-13 and Table 5-8 lists typical Minor loss coefficients for various inlet and manhole configurations:

$$H_{1/M} = \frac{V_2^2 - KV_1^2}{2g}$$ **Equation 5-13**

where: $H_{I/M}$ = loss at inlet or manhole (feet);
K = the Minor loss coefficient, from Table 5-8; V_1 = velocity in the upstream pipe (feet per second);
V_2 = velocity in the downstream pipe (feet per second); and,
g = the acceleration of gravity, 32.2 feet/second^2.

Table 5-8. Minor Loss Coefficients for Inlets and Manholes

Type of Structure	**Coefficient (K)**
Inlet on Main Line	0.50
Inlet on Main Line with Branch Lateral	0.25
Manhole on Main Line with 22.5-Degree Lateral	0.75
Manhole on Main Line with 45-Degree Lateral	0.50
Manhole on Main Line with 60-Degree Lateral	0.35
Manhole on Main Line with 90-Degree Lateral	0.25
Manhole on Main Line with No Change in Pipe Size	0.05

5.5.4.4 Hydraulic Grade Line

As indicated previously, the HGL shall be maintained at an elevation below the top-of-curb at inlets and below the rim elevation at manholes during the design storm. The HGL elevation shall be calculated at each node (i.e., inlet or manhole) in the storm sewer system to ensure that this level of protection is met. For additional information on this level of protection criteria, refer to Chapter 2.

The tailwater elevation in the receiving channel or storm sewer system shall be determined using approved engineering analysis, or by using flood study model HGL elevations, if available.

DRAFT

$$HGL_u = HGL_d + h_f + h_m$$ **Equation 5-14**

where: HGL_u = HGL at upstream end (node) of storm sewer segment (feet);
HGL_d = HGL at downstream end (node) of storm sewer segment (feet);
h_f = friction loss (feet); and
h_m = sum of Minor losses (feet).

The HGL at the downstream end (node) of the next storm sewer segment upstream can be estimated as the upstream HGL calculated for the downstream segment of storm sewer from this node or the top of the upstream pipe, whichever is greater. For example, in Figure 5-23 the HGL at node 1 would be equal to the WSEL in the receiving system. The upstream HGL for pipe *A* at node 2 would be estimated using Equation 5-14, while the downstream HGL for pipe B at node 2 would be equal to the top of pipe (B) at node 2 or the upstream HGL for pipe A at node 2, whichever is greater. Equation 5-14 would be used to determine the upstream HGL for pipe B at node 3.

The HGL computations should proceed in an upstream direction into all branches of the storm sewer system. If the HGL elevation is above the top-of-curb at any inlets or the rim elevation at any manholes, the storm sewer system must be adjusted so that the HGL does not exceed these elevations. This can typically be accomplished by increasing the capacity of the storm sewer segments with the most significant losses. However, adjustments to storm sewer flowline elevations may also be required.

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Chapter 6
PAVEMENT DRAINAGE, ROADSIDE DITCHES AND INLETS

6.1 PAVEMENT DRAINAGE

6.1.1 Design Objectives

A chief objective in the design of a storm drain system is to move accumulated water off the roadway as quickly and efficiently as possible. Where the flow is concentrated, the design objective shall be to minimize the depth and extent of that flow.

Appropriate longitudinal and transverse slopes assist with moving water off the travel way to minimize ponding, sheet flow, and low crossovers. Therefore, the storm sewer and roadway for a project shall be designed as an integrated system to assure efficient drainage in accordance with the geometric and pavement design.

Since roadways also function as extreme event overland flow paths, they must be considered during the drainage system design.

6.1.2 Ponding

The flow of water in the gutter shall be restricted to a depth and corresponding width that will not pose a hazard to traffic and pedestrians. The ponding/depth of flow criteria outlined in Chapter 2 shall be adhered to in roadway design. The depth of flow depends on the following:

a. rate of flow
b. longitudinal gutter slope
c. transverse roadway slope
d. roughness characteristics of the gutter and pavement
e. inlet spacing
f. adverse hydraulic impacts of speed bumps

Inlets shall be placed at all low points in the roadway surface and at suitable intervals along extended gutter slopes as necessary to prevent excessive ponding on the roadway. In order to minimize the number of inlets and associated costs, the ponded width/depth shall be allowed to approach the limit specified. In instances such as a narrow shoulder or low grades, a continuous removal of flow from the surface may be required. Section 6.3 provides additional information on inlets and spacing requirements. Longitudinal gutter slopes shall not be less than 0.3% (0.003 ft/ft); longitudinal gutter slopes in cul-de-sacs shall not be less than 0.4% (0.004 ft/ft); and transverse pavement slopes for the outer lane shall not be less than 2.0% (0.02 ft/ft). When utilized, speed bumps shall be located such that their effect upon street ponding is minimized or mitigated by using additional inlets.

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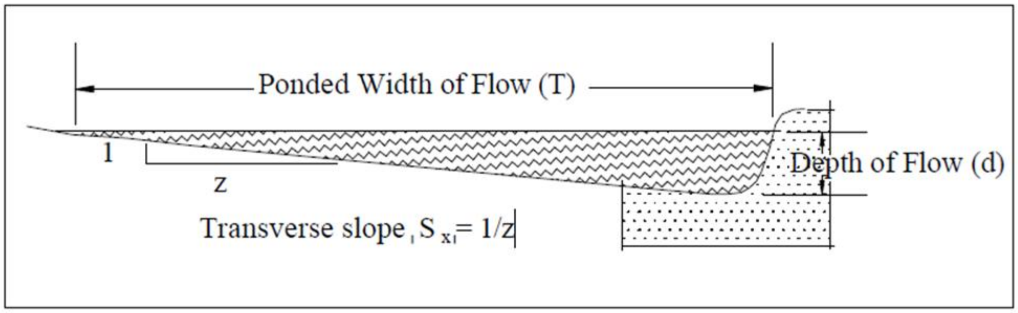
6.1.3 Gutter Flow Design Equations

Figure 6-1 illustrates ponding spread. Ponded width is commonly designated as T.

Figure 6-1. Gutter Flow Cross Section Definition of Terms



The ponded width is a geometric function of the depth of the water (d) in the curb and gutter section. For storm drain system design, the depth of flow in a curb and gutter section with a longitudinal slope (S) is taken as the uniform (normal) depth of flow. As discussed in Section 3.2.1, Manning's Equation can be used to solve for d. Although, it is not possible to solve for uniform depth of flow directly from Manning's Equation, the portion of wetted perimeter represented by the vertical (or near-vertical) face of the curb is ignored in Equation 6-1. However, this assumption does not significantly alter the resulting estimate of d:

$$d = 1.24\left(\frac{QnS_x}{S^{1/2}}\right)^{3/8}$$ **Equation 6-1**

where: d = depth of water in the curb and gutter cross section (ft);
Q = gutter flow rate (cfs);
n = Manning's roughness coefficient;
S = longitudinal slope (ft/ft); and,
S_x = pavement cross slope (ft/ft).

Refer to Figure 6-1, and translate the depth of flow to a ponded width on the basis of similar triangles.

The ponded width in a sag configuration can be determined with Equation 6-2 using depth of standing water or head on the inlet in place of d:

$$T = \left(\frac{d}{S_x}\right)$$ **Equation 6-2**

where: T = ponded width (ft)

DRAFT

Equations 6-1 and 6-2 can be combined to compute the gutter capacity using Equation 6-3:

$$Q = \left(\frac{0.56}{n}\right) S_x^{5/3} S^{1/2} T^{8/3}$$ **Equation 6-3**

Rearranging Equation 6-3 gives a solution for the ponded width (T):

$$T = 1.24 \left(\frac{Qn}{S_x^{5/3} S^{1/2}}\right)^{3/8}$$ **Equation 6-4**

Table 6-1 presents Manning's n values for various pavement surfaces.

Table 6-1. Manning's n Values for Street and Pavement Gutters

Pavement Type	n Value
Asphalt	0.020
Concrete	0.022

6.1.4 Ponding on Continuous Grades

To ensure the ponding depth remains below the top of curb, place storm drain inlets at required intervals. See Section 6.3 for additional information on inlets to determine the gutter ponding at a specific location, such as an inlet, on a continuous grade using the following steps:

a. Determine the total discharge in the gutter based on the drainage area to the desired location.

b. Determine the longitudinal slope and cross-section properties of the gutter. Cross-section properties include transverse slope and Manning's roughness coefficient.

c. Compute the ponded depth using Equation 6-1 and the ponded width using Equation 6- 2.

d. Ponding calculations should be made with software such as Bentley Flowmaster, Geopak, OpenRoads Designer, Hydraflow Storm Sewer, or other equivalent programs.

6.1.5 Ponding at Approach to Sag Locations

The sag inlet capacity, flow in the gutter approaching the left side of the sag inlet, and flow in the gutter approaching the right side of the sag inlet, should be considered at sag locations. The procedure outlined below can be used to avoid exceeding allowable ponding:

DRAFT

a. Estimate the portion of runoff contributing to the left and right approaches by computing the discharge to the sag location based on the entire drainage area and determining the approximate fraction of area contributing to each side of the sag location. Then, each fraction can be multiplied by the total discharge to determine the discharge to each side.

b. Determine the longitudinal slope of each gutter approach. For sawtooth profiles, the slopes will be the profile grades of the left and right approaches. However, if the sag is in a vertical curve, the slope at the sag is zero, which would mean that there is no gutter capacity. In reality there is a three-dimensional flow pattern resulting from the drawdown effect of the inlet. The longitudinal slope can be approximated as fifty-percent of the tangent grade.

c. Calculate the ponded depth and width for each side of the sag using the appropriate flow apportionment, longitudinal slope, and Equation 6-1. Compute the ponded width using Equation 6-2.

d. Ponding calculations should be made with software such as Bentley Flowmaster, Geopak, OpenRoads Designer, Hydraflow Storm Sewer, or other equivalent programs.

6.2 DESIGN OF ROADSIDE DITCHES

6.2.1 General Requirements for Roadside Ditches

Roadside ditches and culverts shall be designed to convey 5-year peak runoff rates at maximum water levels not to exceed top of bank elevations.

6.2.2 Design Flow Rates for Roadside Ditches

Design peak flow rates will typically be computed using the Rational Method.

6.2.3 Design Requirements for Roadside Ditches

Roadside ditches and culverts shall be designed to convey 5-year peak runoff rates at or below top of bank elevations.

The following requirements shall be applied to the designs of all roadside ditches.

Side Slopes:	3:1 (H:V) or flatter
Freeboard:	Not required
Minimum Velocity:	2 fps (flowing full)
Erosion Protection:	Establish vegetative growth (top soil and/or erosion mats may be required)

NOTE: Drainage easement will be required if ditch cannot be contained within the road

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right-of-way.

a. Roadside ditches shall be designed with side slopes in accordance with TxDOT design speed criteria where City criteria does not apply.

b. The minimum grade for roadside ditches shall be 1.0% where practical.

c. The minimum culvert size for roadside ditches shall be 18 inches.

d. The Engineer shall ensure that the carrying capacity of culverts is equal to or greater thanthe carrying capacity of the ditch.

6.3 INLETS

6.3.1 Introduction

Inlets used for roadway drainage can be divided into three classes:

a. Curb opening inlets (*Figure 6-2*)

b. Grate inlets (Figure 6-4)

c. Combination inlets – Combination inlets consist of a curb-opening inlet and a grate inlet. In a curb and grate combination, the curb opening may extend upstream of the grate.

The following requirements shall be applied to the design of inlets:

a. Inlets shall be placed at all low points (sag inlets) in the roadway surface and at suitable intervals along extended gutter slopes (on-grade inlets) as necessary to prevent excessiveponding on the roadway (as defined by Table 2-1).

b. Grate inlets are discouraged, but combination inlets are allowed.

c. Inlet type, size and spacing shall be properly designed so as <u>not to exceed</u> the ponding and/or spread of water limits as shown in Table 2-1.

6.3.2 Curb Opening Inlets

Figure 6-2 illustrates a generic example of a typical curb opening inlet. Curb inlets are used in urban roadway sections along the curb line on continuous grades (on-grade) and at sag locations.

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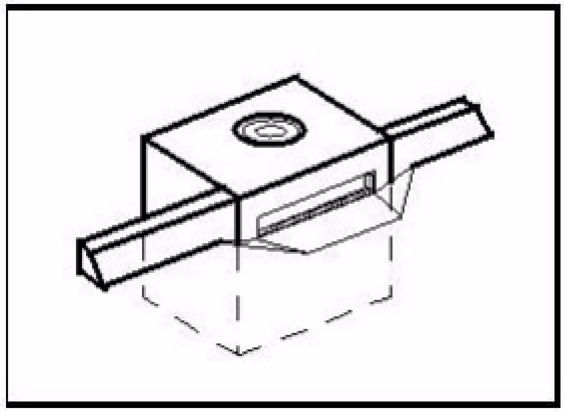
Figure 6-2. Curb Opening Inlet

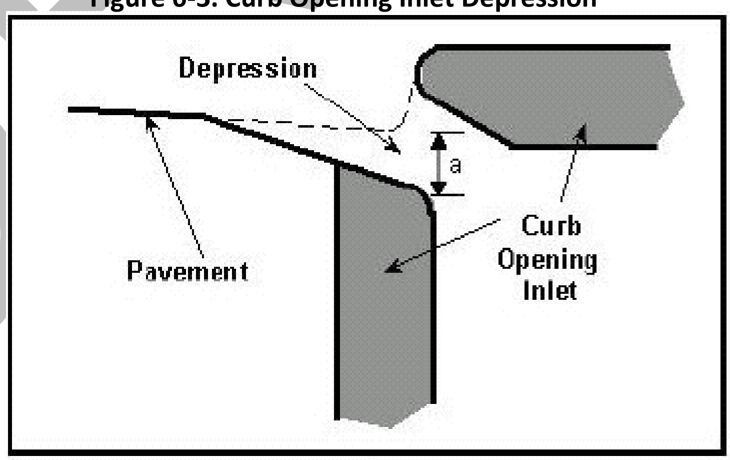


As shown in Figure 6-3, most curb opening inlets depend heavily upon an adjacent depression in the gutter for effective flow interception. Greater interception rates result in shorter and more economical inlet lengths. However, a large gutter depression can be unsafe for traffic flow moving near the gutter line. Therefore, it is necessary to balance safety and economy by selecting an appropriate value for the gutter depression. Refer to City standard construction details for typical inlet gutter dimensions. If city standards are not attainable, then the depth of the gutter depression should be:

a. 0 to 1 inches where the gutter is within the traffic lane

b. 1 to 3 inches where the gutter is outside the traffic lane or in the parking lane

Figure 6-3. Curb Opening Inlet Depression



Curb opening inlets are useful in sag and on-grade situations because of their self-cleansing abilities and hydraulic efficiency. Additionally, they are often preferred over grate inlets because the inlet is placed outside the travel way and poses less of a risk to motorists and bicycle traffic.

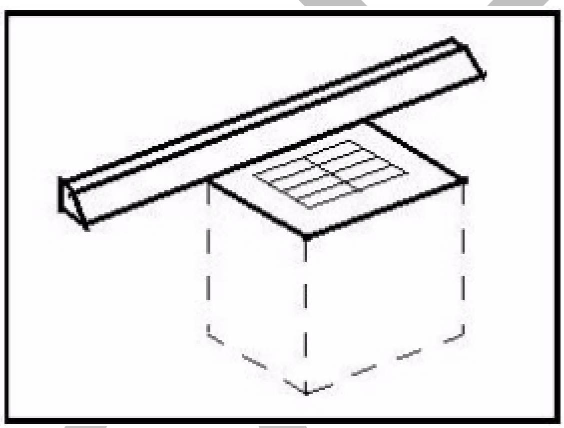
DRAFT

A drawback of curb opening inlets is that the flowline of the opening is fixed and not readily adaptable to changing pavement levels as occur in surface treatment overlays. Successive overlays can gradually reduce or even eliminate the original opening available for water removal, unless the pavement edge is tapered to the original gutter line. Pavement should be milled prior to overlays to avoid reducing gutter capacity.

6.3.3 Grate Inlets

Figure 6-4 illustrates a typical grate inlet. In these types of inlets, water falls into the inlet through a grate instead of an opening in the curb. There are many variations of this inlet type, and the format of the grate itself varies widely as each foundry may have its own series of standard fabrication molds.

Figure 6-4. Grate Inlet Schematic



Grate inlets should be an exception and mainly used under the following circumstances:

- Sag configurations in gutters adjacent to concrete traffic barriers or rails where curb inlets would not be practicable;
- V-shaped gutters with no curb or barrier;
- Ditches; and
- On-grade situations with curb inlets.

However, when they are intended to intercept gutter flow in an on-grade configuration, the grate openings should be oriented parallel to the gutter flow in order to maximize hydraulic efficiency.

When necessary, for residential development, a combination inlet (Section 6.3.4) shall be used. Grate inlets are intended for use in urban roadway features such as driveways, street intersections, and medians. It is important to assure that the grate configurations and orientations are compatible with bicycle and wheelchair safety.

One important benefit of grate inlets is that they can be removed to provide excellent

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access to the storm drain system. However, their tendency to collect debris can result in significant maintenance requirements. As debris accumulates, it obstructs the flow of water into the inlet.

6.3.4 Combination Inlets

Combination curb and grate inlets can be useful in many configurations, especially in sag locations. Because of the inherent debris problem in sags, the combination inlet offers an overflow drain if part of the inlet becomes completely or severely clogged by debris. Maintenance of combination inlets is usually facilitated by the removable grate, which provides easy access to the inlet and associated storm drain system.

For a combination curb and grate, assume that the capacity of the combination inlet comprises the sum of the capacity of the grate (assuming 50% clogging) plus the capacity of the curb opening.

6.3.5 Inlets in Sag Configurations

Inlets are required at all sag configurations, or low points in the pavement. An inlet in a sag configuration is the "end of the line" because the water and its debris load have no other place to go other than an associated extreme event overflow corridor. Because of this, failure of an inlet in a sag configuration often represents a threat to the successful operation of a storm drain system. In a sag configuration, the controlling ponded width can be from one of three origins.

a. The inlet itself may cause a head that translates to a ponded width.

b. As water approaches the sag configuration inlet from each of two directions, the flow in the curb and gutter from each direction subtends its own ponded width.

c. If the sag configuration inlet is in the trough of a vertical curve, the slope in the immediate vicinity of the sag inlet is equal to 0%. Therefore, no specific slope is available for the computation of gutter flow characteristics. If the low point inlet is located at the intersection of two tangent approach slopes with no vertical curve, use the actual longitudinal slopes for the calculation of flow depths in the gutter.

Because the water or its debris load can go no place other than an associated extreme event overflow corridor, an appropriate safety factor shall be applied to the inlet size. For grate inlets in sags, the usual safety factor is approximately two. For curb inlets, the ratio can be somewhat less. For example, if a low point grate inlet requires an open area of 4.1 ft^2 and the standard inlet open area is 4.0 ft^2, provide two inlets for a total open area of 8.0 ft^2 (safety factor = 1.9).

In addition, where significant ponding can occur such as in underpasses and in sag-vertical curves, it is good engineering practice to place flanking inlets on each side of the sag location inlet. Analyze flanking inlets as inlets on-grade at some specified distance away from the low point on the sag vertical curve. Often, the specified distance is 50 or 100 ft.

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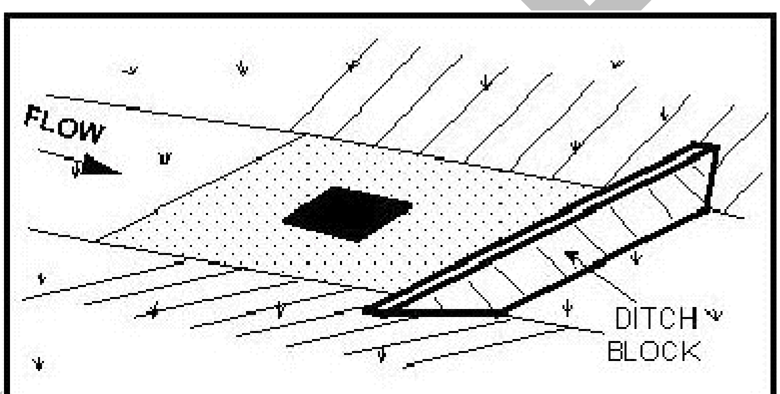
The on-grade inlets serve to relieve some or most of the flow burden from the inlet located at the low point. Place the flanking inlets so that they will limit spread on low gradient approaches to the level point and act in relief of the sag inlet if it becomes clogged or if the design spread is exceeded.

6.3.6 Median/Ditch Drains

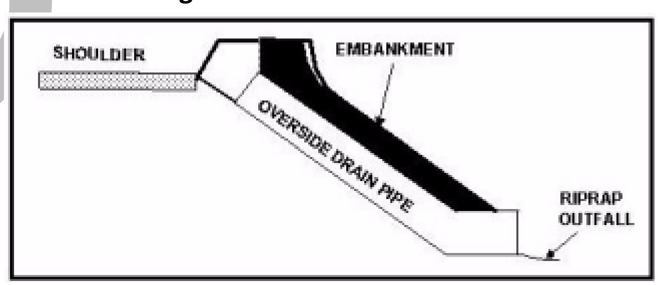
Drains or inlets appearing in ditches and medians are usually termed "drop inlets" or "post inlets". Often, such an inlet is in a sag (sump) configuration. In sag configurations, drains have a high probability of maintenance problems. As with grate inlets in gutters, grate inlets used in medians or other ditches should usually have the grate bars aligned parallel to the flow. A concrete slab that forms a type of bowl around the inlet may improve the operational characteristics of the facility. If the inlet in the median or ditch is in an on-grade configuration, a downstream dike or "ditch block" may need to be provided as illustrated in Figure 6-5.

Figure 6-5. Median/Ditch Inlet



Over-side drains, also referred to as drainage chutes, may be used when no inlet at the curb and gutter line connects to a storm drain system. An opening in the curb connecting to a scour resistant channel or chute removes the concentrated flow in the curb and gutter from the roadway. In some instances, the channel or chute may be replaced with a small pipe placed in the roadway embankment as illustrated in Figure 6-6.

Figure 6-6. Over-Side Drains



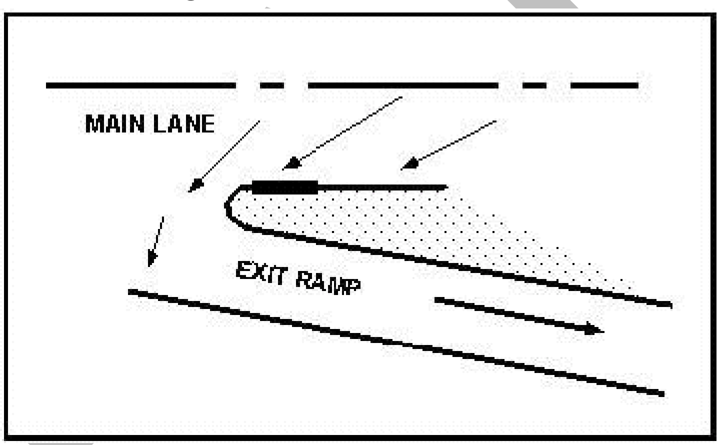
6.3.7 Inlet Design

6.3.7.1 Inlet Locations

The inlet location shall be dictated on the basis of physical demands and hydraulic requirements. In all instances, the inlet location must be coordinated with physical characteristics of the roadway geometry, utility conflicts, and feasibility of underground pipe layout.

Establish logical locations early on as permanent and non-adjustable fixtures in the storm drain system. Determine their hydraulic characteristics in the ordinary trial and error process of storm inlet design as illustrated in Sections 6.3.7.2 through 6.3.7.9. Logical locations for inlets include near street intersections, at gore islands (see Figure 6-7), and at super-elevation transitions. Note - inlets must be provided at all low points (sag configurations) in the pavement.

Figure 6-7. Inlet at a Gore Island



6.3.7.2 Ponded Depth Options

An on-grade inlet may be necessary to remove some or all of the flow at that point so that the basic design criterion, allowable ponded depth, is not violated. For a given tentative inlet location, determine the ponded depth to that point. Figure 6-8 shows interdependence of inlet location, drainage area, discharge, and ponded width. If the calculated ponded depth is greater than the allowable ponded depth, there are two options:

a. Relocate the inlet to a point upstream in the curb and gutter section. This reduces the watershed area and the associated peak discharge. The lowered peak discharge causes a lower ponded depth. This results in an increased drainage area at the next downstream location, thus increasing the discharge and ponding.

b. Locate an intermediate inlet at some point upstream in the curb and gutter section. This intermediate inlet defines a new watershed from which a reduced discharge flows, which reduces the ponded depth at the original inlet location.

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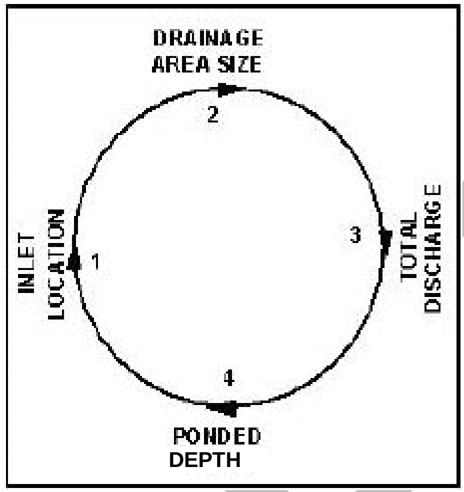
Figure 6-8. Relation of Inlet Location to Design Discharge



If the calculated ponded depth is less than or equal to the allowable ponded depth, then the calculated ponded depth can be compared to the allowable ponded depth to determine if it represent an efficient design. If all or most of the allowable ponded depth is used, the location is probably efficient. If only a small portion of the allowable ponded depth is used, then a more efficient location may be possible. In extensive storm drain systems, minimizing the number of inlets should be a design objective. This can be accomplished by using as much of the allowable ponded depth as is possible.

6.3.7.3 Carryover Design Approach

Intercepting a portion of the total gutter flow with an on grade inlet is more efficient than trying to intercept the entire gutter flow. The rate of gutter flow not intercepted is called “carryover”. This design approach is recommended in those instances where it is not necessary to intercept all of the flow and can only be applied to on-grade inlet configurations.

Figure 6-9 illustrates an inlet that is designed to intercept all of the approaching flow. Note the large portion of inlet opening that is not utilized efficiently.

Figure 6-9. Inlet Designed with No Carryover

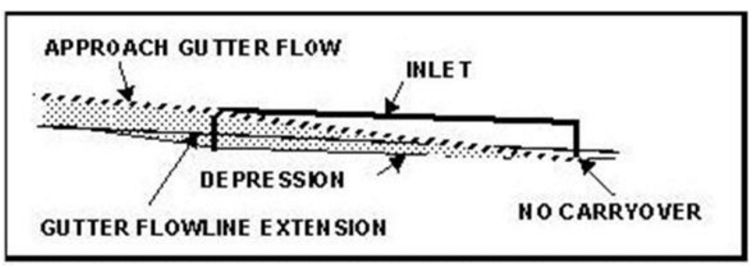


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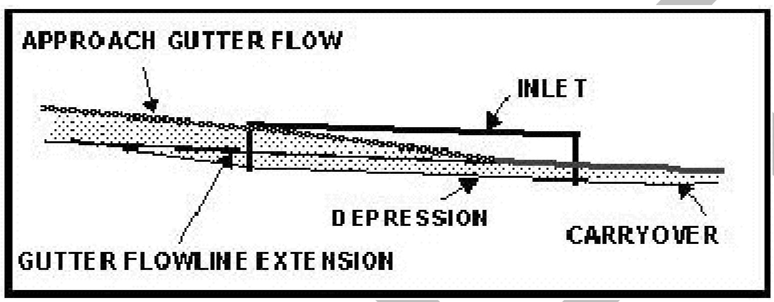
Figure 6-10 illustrates the concept of flow carryover. Note that the inlet opening is used much more efficiently for flow interception than the inlet illustrated in Figure 6-9.

Figure 6-10. Inlet Designed with Carryover



Carryover flows must be accommodated by ultimate interception at some other location (sometimes termed “bypass flow”). Furthermore, the gutter between the two points must accommodate the additional carryover rate. Carryover is not recommended upstream of street intersections and driveways, at super-elevation transitions where the cross slope begins to reverse, and below entrance/exit ramps onto major highways.

Software programs such as Bentley Geopak and OpenRoads Designer, Hydraflow Storm Sewer, or their equivalent, should be used to calculate inlet bypass (carryover) throughout the system being analyzed.

6.3.7.4 Curb Inlets On-Grade

The design of on-grade curb opening inlets requires determination of length needed for total flow interception, a subjective decision about the actual length to be provided, and determination of any resulting carryover rate.

For each on-grade inlet, determine early whether or not carryover is to be a valid design consideration. In some cases, due to a logical location of the inlet, no carryover may be allowed. In other cases, where carryover is acceptable, there may not be a convenient location to accommodate the bypass flow.

Use the following procedure to design curb inlets on-grade:

a. Compute depth of flow and ponded width (T) in the gutter section at the inlet using Equations 6-1 and 6-2.

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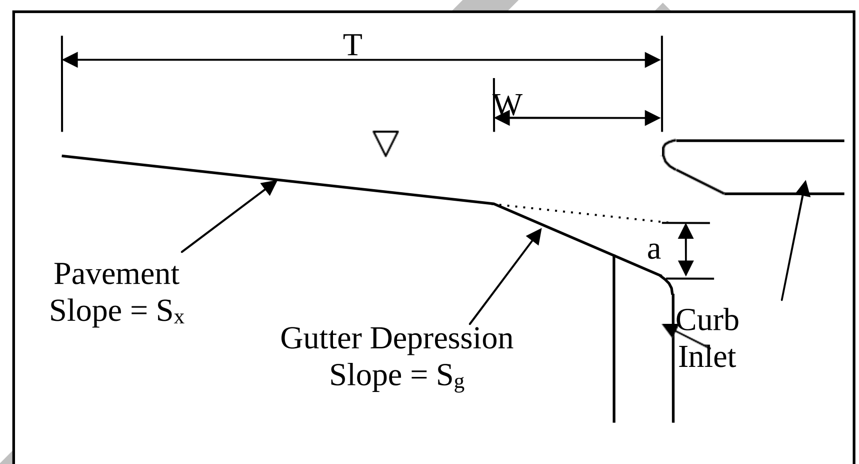
b. Determine the ratio of the width of flow in the depressed section (W) to the width of total gutter flow (T) using Equation 6-5. Figure 6-11 shows the gutter cross section at an inlet.

$$E_o = \frac{K_w}{K_w + K_o}$$ **Equation 6-5**

where: E_o = ratio of depression flow to total flow;
K_w = conveyance of the depressed gutter section (cfs); and,
K_o = conveyance of the gutter section beyond the depression (cfs).

Figure 6-11. Gutter Cross-Section Diagram



Use Equation 6-6 to calculate conveyance (K_w and K_o).

$$K_{w,o} = \frac{1.486A^{5/3}}{nP^{2/3}}$$ **Equation 6-6**

where: $K_{w,o}$ = conveyance of cross section (cfs);
A = area of cross section (ft^2);
n = Manning's roughness coefficient; and
P = wetted perimeter (ft).

Use Equation 6-7 to calculate the area of cross section in the depressed gutter section (A_w).

$$A_w = WS_x\left(T - \frac{W}{2}\right) + \frac{1}{2}aW$$ **Equation 6-7**

where: A_w = area of depressed gutter section (ft^2);
W = depression width for an on-grade curb inlet (ft);
S_x = cross slope (ft/ft);

DRAFT

T = calculated ponded width (ft); and
a = curb opening depression depth (ft).

Use Equation 6-8 to calculate the wetted perimeter in the depressed gutter section (P_w).

$$P_w = \sqrt{(WS_x + a)^2 + W^2}$$ **Equation 6-8**

where: P_w = wetted perimeter of depressed gutter section (ft²);
W = depression width for an on-grade curb inlet (ft);
S_x = cross slope (ft/ft); and,
= curb opening depression depth (ft).

Use Equation 6-9 to calculate the area of cross section of the gutter section beyond the depressions (A_o).

$$A_o = \left(\frac{S_x}{2}\right)(T - W)^2$$ **Equation 6-9**

where: A_o = area of gutter/road section beyond the depression width (ft²);
S_x = cross slope (ft/ft);
W = depression width for an on-grade curb inlet (ft); and,
T = calculated pond width (ft).

Use Equation 6-10 to calculate the wetted perimeter of the gutter section beyond the depression (P_o).

$$P_o = T - W$$ **Equation 6-10**

where: P_o = wetted perimeter of the gutter section beyond the depression (ft²);
T = calculated pond width (ft); and,
W = depression width for an on-grade curb inlet (ft).

c. Use Equation 6-11 to determine the equivalent cross slope (S_e) for a depressed curb opening inlet.

$$S_e = S_x + \left(\frac{a}{W}\right)E_o$$ **Equation 6-11**

where: S_e = equivalent cross slope (ft/ft);
S_x = cross slope of the road (ft/ft);
a = gutter depression depth (ft);
W = gutter depression width (ft); and,
E_o = ratio of depression flow to total flow.

DRAFT

d. Calculate the length of curb inlet required (L_r) for total interception using Equation 6-12.

$$L_r = (0.6Q^{0.42}S^{0.3})\left(\frac{1}{nS_e}\right)^{0.6}$$ **Equation 6-12**

where: L_r = length of curb inlet required (ft);
Q = flow rate in gutter (cfs);
S = longitudinal slope (ft/ft);
n = Manning's roughness coefficient; and
S_e = equivalent cross slope (ft/ft).

If no carryover is allowed, the inlet length is assigned a nominal dimension of at least L_r. Use a nominal length available in standards for curb opening inlets. Do not use the exact value of L_r if doing so requires special details, special drawings and structural design, andcostly and unfamiliar construction. If carryover is considered, round the curb opening inlet length down to the next available (nominal) standard curb opening length and compute the carryover flow.

e. Determine carryover flow using Equation 6-13. In carryover computations, efficiency of flow interception varies with the ratio of actual length of curb opening inlet supplied (L_a) to length L_r and with the depression to depth of flow ratio.

$$Q_{co} = Q\left(1 - \frac{L_a}{L_r}\right)^{1.8}$$ **Equation 6-13**

where: Q_{co} = carryover discharge (cfs);
Q = total discharge (cfs);
L_a = design length of the curb opening inlet (ft); and,
Lr = length of curb opening inlet required to intercept the total flow (ft).

Carryover rates usually should not exceed 0.5 cfs, or about 30% of the original discharge. Greater carryover rates can be problematic and cause a significant departure from the principles of the Rational Method application. In all cases, the carryover rate must be accommodated at some other specified point in the storm drain system.

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f. Calculate the intercepted flow (Q_i) using Equation 6-14. Calculate the intercepted flow as the original discharge in the approach curb and gutter minus the amount of carryover flow.

$$Q_i = Q - Q_{co}$$ **Equation 6-14**

6.3.7.5 Curb Inlets in Sag Configuration

The capacity of a curb inlet in a sag depends on the water depth at the curb opening and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage and the capacity shall be based on the lesser of the computed weir and orifice capacity. Generally, this ratio should be less than 1.4 such that the inlet operates as a weir.

a. If the depth of flow in the gutter (d) is less than or equal to 1.4 times the inlet opening height (h) ($d \leq 1.4h$), determine the length of inlet required considering weir control. Calculate the capacity of the inlet when operating under weir conditions with Equation 6-15. If $d > 1.4h$, skip this step.

$$Q_w = C_w L d^{1.5}$$ **Equation 6-15**

Rearrange Equation 6-15 to produce the following relation for curb inlet length required.

$$L = \frac{Q}{(C_w d^{1.5})}$$ **Equation 6-16**

where: Q = total flow reaching inlet (cfs);
C_w = weir coefficient ($ft^{0.5}/s$), suggested value = 2.3 $ft^{0.5}/s$;
d = head at inlet opening (ft) computed with Equation 6-1; and
L = length of curb inlet opening (ft).

b. If the depth of flow in the gutter (d) is greater than the inlet height (h) ($d > h$), determine the length of inlet required considering orifice control using Equation 6-17.

$$Q = C_o h L \sqrt{2gh}$$ **Equation 6-17**

where: Q = total flow reaching inlet (cfs);
C_o = orifice coefficient = 0.67;
h = depth of opening (ft) (this depth will vary slightly with the inlet detail used);

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L = length of curb opening inlet (ft);
g = acceleration due to gravity = 32.2 ft/s^2; and
d_e = effective head at the centroid of the orifice (ft), $d_e = d - h/2$.

Rearranging Equation 6-17 allows a direct solution for required length.

$$L = \frac{Q}{C_O h \sqrt{2 g d_e}}$$ **Equation 6-18**

c. If both steps 1 and 2 were performed (i.e., $h < d \le 1.4h$), choose the larger of the two computed lengths as being the required length.

d. Select a standard inlet length that is greater than the required length.

6.3.7.6 Grate Inlets On-Grade

The capacity of a grate inlet on-grade depends on its geometry and cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness.

The depth of water next to the curb is the main factor affecting the interception capacity of grate inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate (frontal flow), is intercepted by grate inlets. A small portion of the flow along the length of the grate, termed side flow, is also intercepted at low velocities. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash over occurs. For grates less than 2 feet long, intercepted flow is small. Agencies and manufacturers of grates have investigated inlet interception capacity. For inlet efficiency data for various sizes and shapes of grates, refer to HEC-22.

6.3.7.6.1 Bicycle Safety for Grate Inlets On-Grade

A parallel bar grate is the most efficient type of gutter inlet. However, crossbars added for bicycle safety reduces the efficiency of the inlet. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended due to their hydraulic capacity and bicycle safety features. In certain locations where leaves may create constant maintenance problems, the parallel bar grate may be used more efficiently if bicycle traffic is prohibited.

6.3.7.6.2 Design Procedure for Grate Inlets On-Grade-grade:

a. Compute the ponded width of flow (T) using the outline provided in Section 6.1.4 “Ponding on Continuous Grades”.

b. Choose a grate type and size.

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c. Find the ratio of frontal flow to total gutter flow (E_o) for a straight cross-slope using

Equation 6-19. No depression is applied to a grate on-grade inlet.

$$E_o = 1 - \left(1 - \frac{W}{T}\right)^{8/3}$$ **Equation 6-19**

Find the ratio of frontal flow intercepted to total frontal flow (R_f) using Equations 6-20, 6-21, 6-22, and 6-23.

$$R_f = 1 - 0.3(v - v_o), if\ v > v_o$$ **Equation 6-20**

$$R_f = 1.0, if\ v < v_o$$ **Equation 6-21**

where: R_f = ratio of frontal flow intercepted to total frontal flow;
v = approach velocity of flow in gutter (ft/sec);
v_o = minimum velocity that will cause splash over grate (ft/sec).

For triangular sections, calculate the approach velocity of flow in gutter (v) using Equation 6-22

$$v = \frac{2Q}{Ty} = \frac{2Q}{T^2 S_x}$$ **Equation 6-22**

Otherwise, compute the section area of flow (A) and calculate the velocity using Equation 6-23

$$v = \frac{Q}{A}$$ **Equation 6-23**

Calculate the minimum velocity (v_o) that will cause splash over the grate using the appropriate equation in Table 6-2.

where: v_o = splash-over velocity (ft/sec); and
L = length of grate (ft).

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Table 6-2. Splash-Over Velocity Equations

Grate Configurations	Typical Bar Spacing (inches)	Splash-over Velocity Equation
Parallel Bars	2	$V_o = 2.218 + 4.03 – 0.649L^2 +0.056L^3$
Parallel Cars	1.2	$V_o = 1.762 + 3.117L – 0.451L^2 + 0.033L^3$
Transverse Curved Vane	4.5	$V_o = 1.381 + 2.78L – 0.0300L^2 + 0.020L^3$
Transverse 45o Titled Vane	4	$V_o = 0.988 + 2.625L – 0.359L^2 + 0.029L^3$
Parallel bars w/ transverse rods	2 parallel/4 trans	$V_o = 0.735 + 2.437L – 0.256L^2 + 0.018L^3$
Transverse 30o Tilted Vane	4	$V_o = 0.505 + 2.344L – 0.200L^2 + 0.014L^3$
Reticuline	N/A	$V_o = 0.030 + 2.278L – 0.179L^2 + 0.010L^3$

d. Find the ratio of side flow intercepted to total side flow (R_s) using Equation 6-24.

$$R_s = \left[1 + \frac{0.15v^{1.8}}{S_x L^{2.3}}\right]^{-1} \qquad \textbf{Equation 6-24}$$

where: R_s = ratio of side flow intercepted to total flow;
S_x = transverse slope (ft/ft);
v = approach velocity of flow in gutter (ft/sec); and
L = length of grate (ft).

e. Determine the efficiency of the grate (E_f) using Equation 6-25.

$$E_f = R_f E_o + R_s(1 - E_o) \qquad \textbf{Equation 6-25}$$

f. Calculate the interception capacity of the grate (Q_i) using Equation 6-26. If the interception capacity is greater than the design discharge, then skip step 8.

$$Q_i = E_f Q = Q\left[R_f E_o + R_s(1 - E_o)\right] \qquad \textbf{Equation 6-26}$$

g. Determine the carryover (CO) using Equation 6-27**.**

$$CO = Q - Q_i \qquad \textbf{Equation 6-27}$$

h. Depending on the carryover, select a larger or smaller inlet as needed. If the carryover is excessive, select a larger configuration of inlet and return to step 3. If the interception capacity far exceeds the design discharge, consider using a smaller inlet and return to step 3.

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6.3.7.7 Grate Inlets in Sag Configurations

A grate inlet in sag configuration operates in weir flow at low ponding depths. A transition to orifice flow begins as the ponded depth increases. Use the following procedure for calculating the inlet capacity:

a. Choose a grate of standard dimensions to use as a basis for calculations.

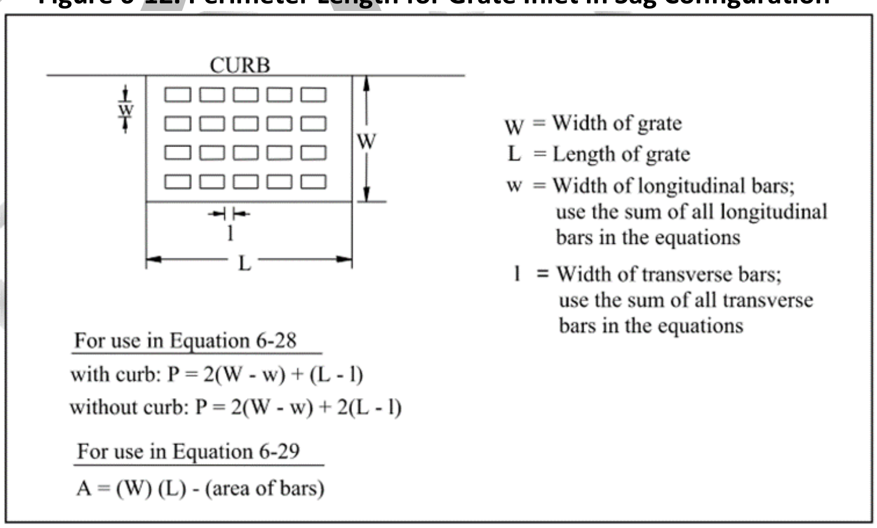
b. Determine an allowable head (h) for the inlet location. This should be the lower of the curb height and the depth associated with the allowable ponded width and/or allowable depth. No gutter depression is applied at grate inlets.

c. Determine the capacity of a grate inlet operating as a weir. Under weir conditions, the grate perimeter controls the capacity. Figure 6-12 shows the perimeter length for a grate inlet located next to and away from a curb. The capacity of a grate inlet operating as a weir is determined using Equation 6-28.

$$Q_w = C_w P h^{1.5}$$ **Equation 6-28**

where: Q_w = weir capacity of grate (cfs);
C_w = weir coefficient, use 3;
P = perimeter of the grate (ft) as shown in Figure 6-12, a multiplier of 0.5 is recommended to be applied to the measured perimeter as a safety factor; and
h = allowable head on grate (ft).

Figure 6-12. Perimeter Length for Grate Inlet in Sag Configuration



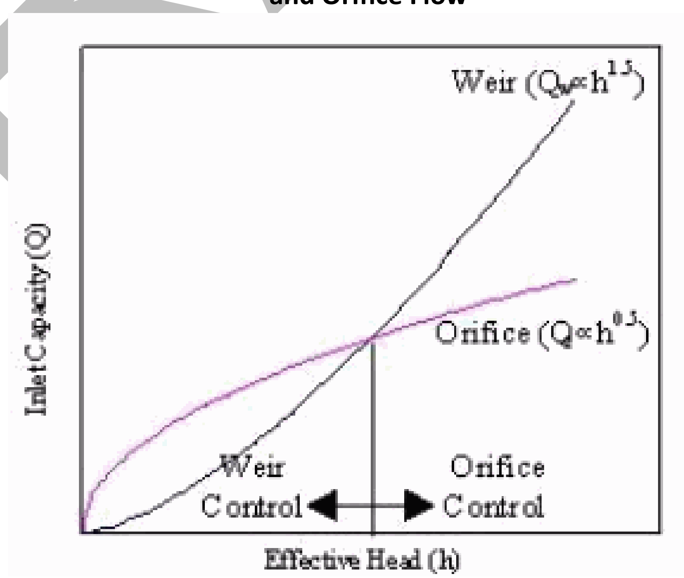
d. Determine the capacity of a grate inlet operating under orifice flow. Under orifice conditions, the grate area controls the capacity. The capacity of a grate inlet operating under orifice flow is computed with Equation 6-29.

$$Q_o = C_o A \sqrt{2gh}$$ **Equation 6-29**

where: Q_o = orifice capacity of grate (cfs);
C_o = orifice flow coefficient, use 0.67;
A = clear opening area of the grate (the total area available for flow) (ft^2), as shown in Figure 6-12; a multiplier of about 0.5 is recommended to be applied to the measured area as a safety factor;
g = acceleration due to gravity (32.2 ft/s^2); and
h = allowable head on grate (ft).

e. Compare the calculated capacities from steps 3 and 4 and choose the lower value as the design capacity. The design capacity of a grated inlet in a sag is based on the minimum flow calculated from weir and orifice conditions. Figure 6-15 demonstrates the relationship between weir and orifice flow. If Q_o is greater than Q_w (to the left of the intersection in Figure 6-13), then the designer would use the capacity calculated with the weir equation. If Q_o is less than Q_w (to the right of the intersection), then the capacity as determined with the orifice equation should be used.

Figure 6-13. Relationship Between Head and Capacity for Weir and Orifice Flow



Chapter 7
DETENTION ANALYSIS

7.1 GENERAL DESIGN REQUIREMENTS

The purpose of this chapter is to provide criteria and guidelines to be used in the analysis of detention facilities. These criteria and guidelines are applicable to both individual on-site detention ponds and regional detention facilities.

Detention facilities are intended to provide storage of excess runoff volume to mitigate increases in peak flows and changes in the timing of runoff associated with urbanization, so that the surrounding and downstream properties and the receiving body of water are not adversely impacted by increases in peak flows or water surface elevations (WSEL). Detention spread throughout a watershed has long term benefits by providing cumulative mitigation of runoff increases over time while improving resiliency by reducing single points of failures. Detention basins can be above or below ground, and may be comprised of excavated ponds, constructed berms, underground tanks, channel storage, or other means.

7.1.1 Detention Requirements

Detention is mandatory for development within the portions of the Oso Creek Watershed in the City limits and Extra Territorial Jurisdiction of the City of Corpus Christi as shown in Figure 7.1. For developments that are not within the mandatory portion of the Oso Creek Watershed, detention will be required if the development will result in an adverse impact, unless said development provides improvements to the downstream storm water infrastructure to offset hydrologic and hydraulic impacts.

Detention requirements for developments are as follows:

a. New development that increases impervious cover.

b. Redevelopment or roadway expansion that increases peak runoff from the site.

c. Drainage system improvements that decrease time of concentration (T_c) or lag time (T_L), or increased conveyance, to the point that peak flows are increased downstream of the site.

d. Timing analysis will not be accepted as a means to avoid providing detention within the mandatory portion of the Oso Creek Watershed.

e. If a development is one single-family residence and not part of a larger development, no detention is required.

f. If a project that does not increase impervious cover and does not increase flows downstream of the site, no detention is required.

The engineer is required to provide all necessary plans, exhibits, and calculations supporting detention requirements. Confirmation of detention requirements and sizing will be made by the City through the platting and permit process.

Detention ponds shall be located within private drainage easement(s) recorded by plat or separate instrument.

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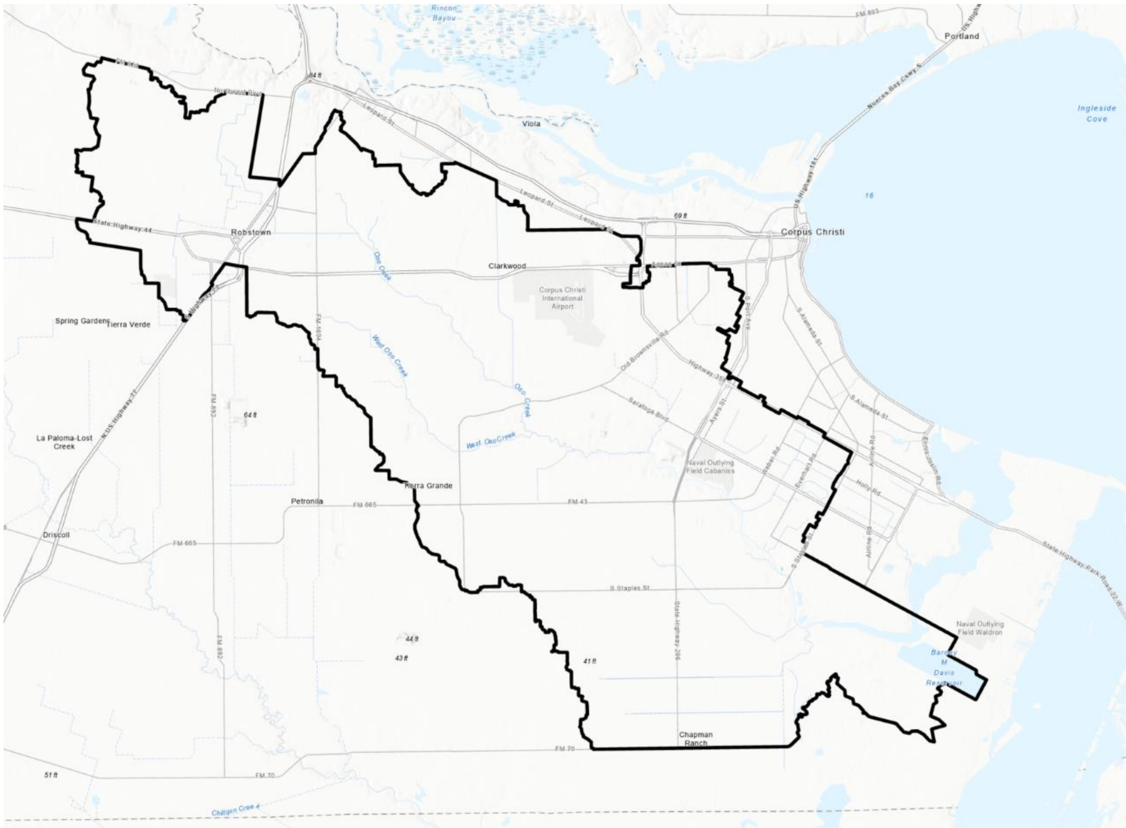
Figure 7-1. Mandatory Detention Area within Oso Creek



7.1.2 Detention Pond Types

Detention Pond designs may vary based on site conditions. Common detention pond types include:

a. **On-site detention**, which is typically within the development property itself. Onsite runoff is typically routed from the site through a basin, and then to the receiving stream.

b. **In-line detention**, which occurs within a channel or large underground system. This would typically occur where a channel or large underground system falls within the subject property.

c. **Off-line detention**, which diverts runoff from a nearby ditch, channel, or drainage system into a pond, typically through an intake weir structure.

d. **Regional detention**, which mitigates runoff from multiple properties through the use of one or more larger detention basins. Regional detention basins shall be maintained by the property owner or property owner association (POA). Ownership and maintenance will not be the responsibility of City.

The City encourages the use of multi-purpose features in detention facilities, provided that the flood risk mitigation and stormwater management function of the facility is not compromised. Water quality design and recreational use may be incorporated into pond design, as long as such design is coordinated with other pertinent City codes, plans, and policies.

Special consideration must be given to detention facilities that incorporate or are within or near FEMA floodplain, wetlands, etc. All federal and state rules and regulations must be followed.

7.2 DESIGN RAINFALL EVENTS FOR DETENTION

Detention design involves providing sufficient storage volume and a restricted outflow structure, designed to mitigate increased runoff for an assortment of design storm events. Detention design will be such that proposed conditions peak flows are reduced to or below existing conditions peak flow for the **5-year, 10-year and 100-year frequency, 24-hour storm events**. Detention basins are generally designed to release runoff and drain over a duration of 24-48 hours.

7.3 SELECTION OF METHOD FOR DETERMINING VOLUME

The appropriate method for determining detention volume depends on the sizes of the contributing watershed and the receiving stream. The required methods are indicated below:

Table 7-1. Detention Method Based on Drainage Area Size

	Contributing Project Area Size (acres)	Calculation Method	Outfall Structure Design Storms
Tier 1	DA < 2	Provide 0.5 acre-feet storage per acre of development OR use the Modified Rational method	5-year, 10-year, and 100-year, 24-hour storm at or below existing conditions
Tier 2	2 < DA ≤ 20	Use the Modified Rational method as long as ponds are not in series	
Tier 3	20 < DA ≤ 200	Use Malcom's Method OR the NRCS Method or another Unit Hydrograph Method	
Tier 4	DA > 200	Use NRCS Method, Green & Ampt, or another Unit Hydrograph Method	

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There are various software programs available to use to size detention ponds. Engineer shall use software acceptable to industry standards and acceptable to the City. Some detention calculations may be performed using proprietary spreadsheets, as long as sufficient information is provided to the City for review of the design. If a project is within, adjacent to, or impacted by the FEMA floodplain, use a FEMA approved software.

Time of concentration (Tc) and lag time (T_L) calculations are critical for hydrologic calculations. T_C and T_L shall be calculated as shown in Chapter 4 Hydrology, or other approved methods selected by the engineer and approved by the City.

Mixing hydrological methods is not allowed. If a small project site is within a large overall watershed that is part of a larger study (e.g., NRCS Method), then the larger watershed method shall be used.

7.3.1 Modified Rational Method

The Modified Rational Method shall be used for detention ponds with drainage areas of 20 acres or less. The Modified Rational Method provides trapezoidal hydrographs for small drainage areas where peak flows may be typically calculated by the Rational Method. These hydrographs show a lower peak flow than calculated with rational method, with the modified peak flow holding the same value over the duration of the peak.

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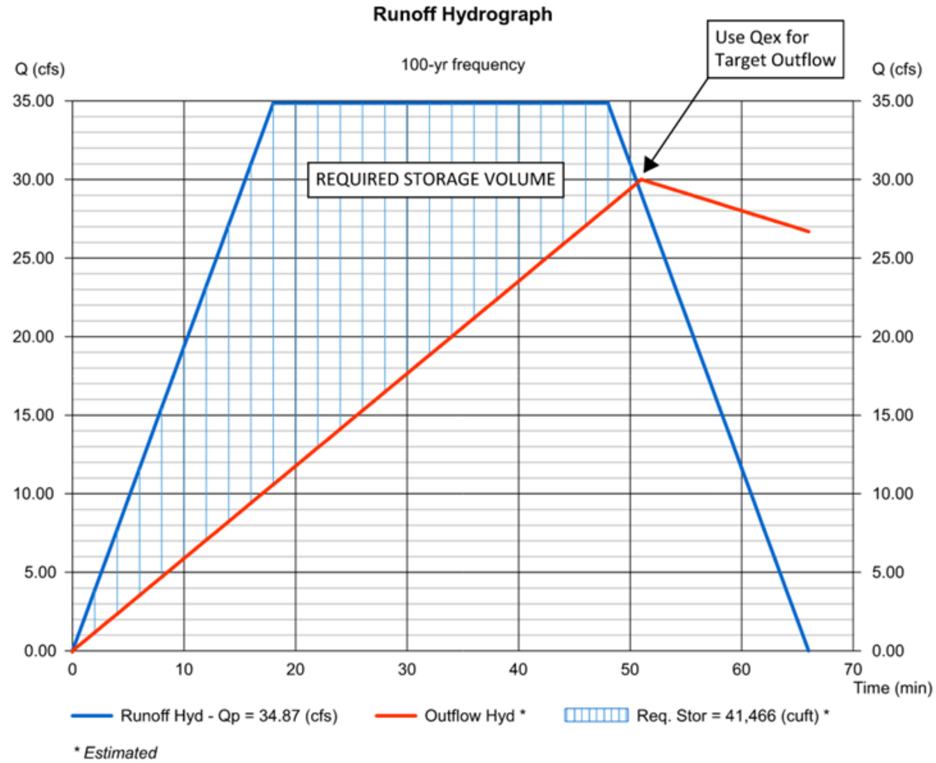
Figure 7-2. Typical Modified Rational Method Hydrograph and Storage Approximation



When sizing a detention pond using modified rational method, first approximate the pond size using modified rational hydrographs only to estimate the storage volume for the 100-year storm event. Then prepare a basic pond size through grading, stage-storage tables, or whatever is suitable for the project. Using that starting pond size, the design is refined in a detention calculation software program or in conjunction with culvert, orifice, and weir calculations.

a. Calculate existing conditions peak flow using rational method. Use Q_{EX} as the target outflow for the modified rational storage calculations.

b. Using a calculation software (Hydraflow Express, Hydraflow Hydrographs, PondPack, Excel, etc.) create a Modified Rational Model using proposed conditions drainage area, Tc, and runoff coefficient, C. For the target outflow, use the Q_{EX} as discussed above.

c. Use the maximum storm duration factor available in the software program. Some programs default to calculating the maximum storm duration by inputting a value of zero (0). Read the software guidelines first. Note: triangular hydrographs will not be accepted for Modified Rational Method results.

d. The storage calculated above is the estimated storage size and may vary from detailed calculations. Input modified rational method hydrology and storage dimensions into a software program (Hydraflow Hydrographs, PondPack, etc.) to refine the design of the pond storage volume and outfall structure.

e. The pond is successfully designed when peak outflow values from the pond are at or below Q_{EX} for the 5, 10, and 100-year storm events, and the storage volume, pond elevations, freeboard, and emergency spillway requirements are met.

7.3.2 Malcom’s Small Watershed Hydrograph Method

The small watershed hydrograph method developed by H.R. Malcom may be used for detention calculations for drainage areas between 20 and 200 acres in size. This method estimates a pattern hydrograph based on the dimensionless hydrograph produced by the Rational Method. It uses a pattern hydrograph peaking at the design flow rate and containing a runoff volume consistent with the design rainfall.

The minimum rate of detention volume using this method shall be 0.50 ac-ft per acre of increased impervious cover for the 100-year storm event.

Malcom’s Method Equations are as follows:

$$T_P = \frac{V}{1.39Q_p}$$ **Equation 7-1**

$$q_i = \frac{Q_p}{2}\left[1 - \cos\left(\frac{\pi t_i}{T_P}\right)\right] \quad for \quad t_i \leq 1.25T_p$$ **Equation 7-2**

$$q_i = 4.34Q_p e^{\frac{-1.30t_i}{T_P}} \quad for \quad t_i > 1.25T_P$$ **Equation 7-3**

Please note the calculator must be in radian mode. The COS function in Excel uses radians by default.

Where, T_P = time (seconds) to Q_P
Q_P = peak design flow rate (cfs) for the drainage area
V = total volume of runoff (ft^3)
t_i = time of interest (seconds)
q_i = flow rate that determines shape of the inflow hydrograph

Obtain Q_P from the Rational Method, found in Chapter 4.

DRAFT

The total volume of runoff, V, is found by applying the total runoff depth (Q) from Equation 4-4 (NRCS Method) to the drainage area, as shown below. This runoff depth is sometimes referred to as rainfall excess. Note that V is not the detention volume.

$$V\ (ft^3) = Q\ (\text{in}) \times Drainage\ Area\ (acre)\ \times \left(\frac{43{,}560\ ft^3/\text{ac-ft}}{12\ \text{in/ft}}\right)$$ **Equation 7-4**

To determine required detention storage for a given storm event, calculate each applicable parameter, then calculated incremental discharge rates at each time step to create pre- and post-project hydrographs. Using the pre- and post-project hydrographs, calculate the cumulative storage. The maximum storage volume represents the required volume for the given storm event, and the corresponding incremental discharge rate represents the maximum allowable peak flow from the detention basin for that storm event.

Table 7-2. Example Calculation of Malcom's Method Parameters

Example Malcom's Method Parameter Caclulations			
Parameter		Pre-Project	Post-Project
Site Info	Drainage Area (ac)	50.00	50.00
	% IC	10.00	38.00
Rational Method Components	Tc (min)	45.00	25.00
	Rainfall Intensity (in/hr)	5.81	7.94
	Runoff Coefficient, C	0.40	0.60
	Peak Flow Qp (cfs)	116	238
NRCS Method Components	24-hr, 100-yr rainfall depth P (inch)	13.30	13.30
	Curve Number, CN	80.00	87.00
	S	2.50	1.49
	Runoff Depth Q (inches)	10.71	11.66
Malcom's Method Parameters	V (cf)	1,943,592	2,116,459
	Tp (sec)	12,033.29	6,392.24
	Tp (min)	200.55	106.54

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Table 7-3. Example of Malcom's Method Calculations to Develop Hydrographs and Determine Detention Volume

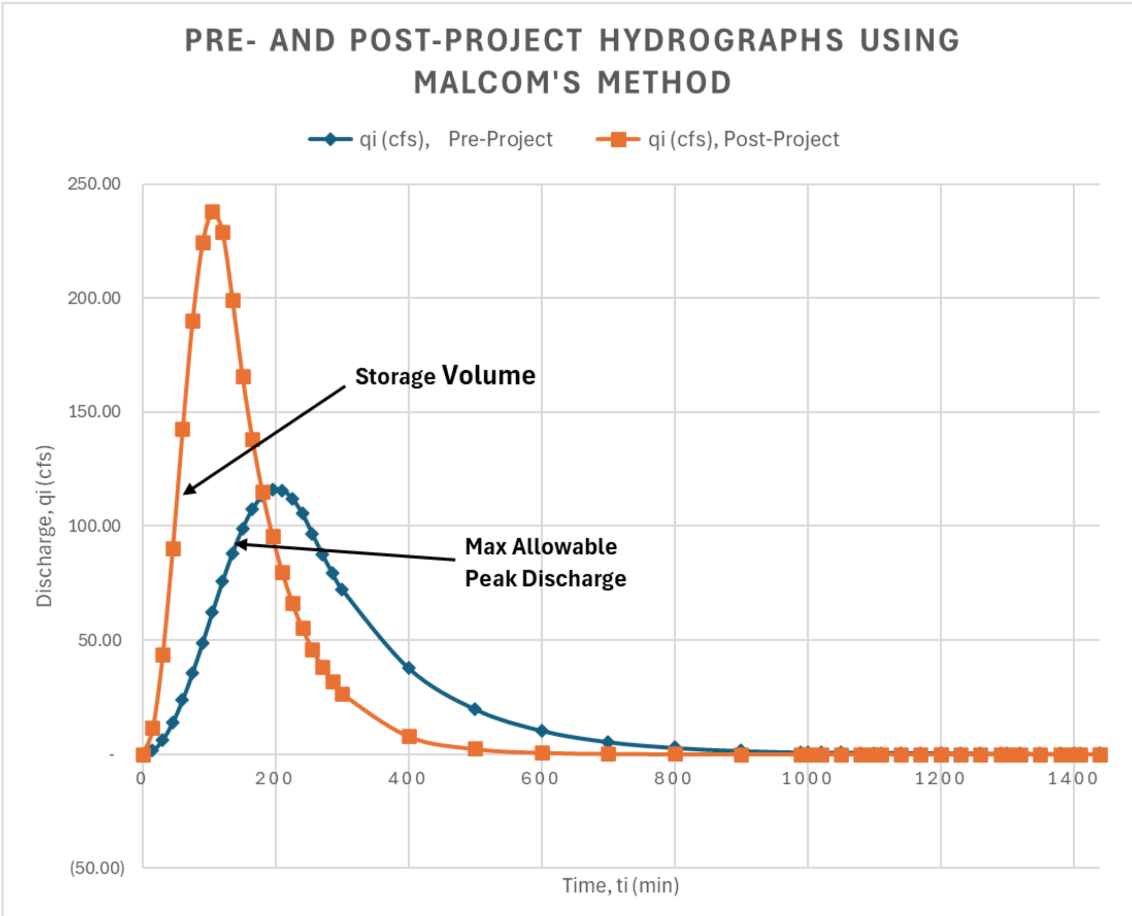
Example Malcom's Method Volume Calculation Table						
		qi (cfs), Pre-Project	qi (cfs), Post-Project	Δqi Post - Pre (cfs)	Volume (ft3) per time step	Volume (ft3) cumulative
0	0	-	-	-	-	-
15	900	1.60	11.46	9.87	8,879.14	8,879.14
30	1800	6.30	43.64	37.34	33,609.90	42,489.04
45	2700	13.85	90.35	76.50	68,850.16	111,339.20
60	3600	23.83	142.59	118.76	106,882.59	218,221.79
75	4500	35.69	190.30	154.61	139,151.39	357,373.18
90	5400	48.79	224.32	175.53	157,977.03	515,350.21
105	6300	62.39	238.08	175.68	158,115.11	673,465.32
120	7200	75.77	228.94	153.17	137,854.83	811,320.15
135	8100	88.17	199.07	110.91	99,817.03	911,137.18
150	9000	98.91	165.78	66.86	60,176.17	971,313.35
165	9900	107.42	138.05	30.63	27,566.39	998,879.75
180	10800	113.21	**114.96**	1.74	1,569.99	**1,000,449.73**
195	11700	115.98	95.73	(20.25)	(18,224.58)	982,225.15
210	12600	115.57	79.72	(35.85)	(32,261.87)	949,963.28
225	13500	111.99	66.38	(45.61)	(41,046.54)	908,916.74
240	14400	105.46	55.28	(50.18)	(45,158.48)	863,758.26

Detention Volume (ft3)
1,000,449.73

Detention Volume (ac-ft)
22.97

Max Allowable Discharge (cfs)
114.96

Figure 7-3. Malcom's Method Hydrographs and Storage in Graphical Form



Similar to the Modified Rational Method, the calculations above provide a starting point to estimate detention. Detention pond design can then be refined with accompanying culvert, orifice, and weir calculations to model the outfall.

7.3.3 Unit Hydrograph Methods

Reference Chapter 4, Hydrology for information related to the NRCS and other Unit Hydrograph Methods.

7.4 OUTFALL STRUCTURE DESIGN

Outfall structures are typically comprised of a primary outfall structure and an emergency/auxiliary outfall structure. The primary structure may be combination of orifices, weirs, and/or risers to restrict flow to meet the peak flow design criteria, and to prevent detention basins from overtopping. Emergency structures serve to protect the basin from overtopping in an uncontrolled manner during a large storm event.

7.4.1 Outfall structure design shall be as follows:

a. Outfall structures situated such that the flow line of the opening matches the flow line of the basin shall be modeled as a culvert (or culvert group for multiple structures).

b. Outfall structures such as a pipe opening may be modeled as orifices if the bottom elevation of the opening is above the flow line of the basin.

c. Outfall weir structures shall be modeled based on the appropriate weir shape (broad-crested, sharp-crested, v-notch, etc.).

d. Point discharges from basin outlet must not discharge directly towards building, into roadways, or onto driveways.

e. Energy dissipation should be designed to reduce velocity at point discharges.

f. Discharge from outflow structures shall not adversely impact properties, structures, or infrastructure downstream.

g. The emergency spillway shall have the capacity to convey the proposed conditions 100-year, 24-hour storm event, without overtopping the pond embankment, assuming the primary outfall structure is clogged.

h. Tailwater conditions must be evaluated to confirm the outfall structure performance is not detrimentally impacted by downstream capacity conditions.

Any detention and detention outfall design not otherwise covered in this manual may be designed in accordance with Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 22 (HEC-22).

Detention outfall structures shall be designed to limit peak discharges to the allowable peak discharge rates. Depending on the detention routing software program, the outfall structure will either be input directly into the model or simulated with a stage-discharge relationship calculated independently and entered into the model. Reference modeling guidelines for the chosen software program to confirm the appropriate steps for modeling the outfall structure.

A constant tailwater elevation in the receiving channel shall be estimated by using a known downstream WSE from an effective FEMA study, an approved City study, or calculated normal depth for each design storm, or as determined by the engineer and approved by the City. This constant tailwater elevation can be used to develop a stage-discharge relationship for an outfall structure. The calculated stage-discharge relationship can then be incorporated into a detention model.

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In areas where tailwater conditions are a concern (i.e., detention basins located in 100-year floodplains, channels where WSELs remain high for long periods, channels with steeply rising WSEs, and areas with existing flooding problems), a stage hydrograph (elevation versus time) shall be developed for the receiving channel and used in a detention modeling software that allows stage hydrographs as a downstream boundary condition.

7.5 DETENTION BASIN DESIGN CONSIDERATIONS

The following are design requirements for detention basins:

a. Detention Ponds shall be designed to drain at least 80% of the 100-year storm volume within 24-48 hours.

b. Maximum vegetated side slopes for ponds shall be 3:1, and 4:1 is recommended where space allows. Maximum concrete or otherwise armored side slopes shall be 2:1.

c. Avoid runoff flowing over the side slopes into the pond. Use inlets to collect runoff above the slope and convey runoff into the pond through a storm drain. Where unavoidable, armor the side slope with erosion control matting, stone or concrete rip rap, or with a manufactured armoring material. Where runoff flows over a concrete side slope, extend the toe down to 36” at the top of the slope.

d. Where possible, avoid berm heights greater than 6’ from the top of berm to the existing grade.

e. All Texas Commission on Environmental Quality (TCEQ) dam safety requirements apply. Refer to the Texas Administrative Code, Title 30, Chapter 299, Subchapter A, Rule §299.1 for further information.

f. An emergency overflow spillway/structure shall be provided to pass the combined 100-year inflow into the pond with 6” of freeboard from top of embankment.

g. Under low flow conditions, basins shall drain completely.

h. Detention basin bottoms must have positive drainage throughout the pond. If pond bottom slopes are less than 1%, slope the pond toward a concrete pilot channel that drains from the pond inlet to the pond outlet.

i. Concrete pilot channels shall be a minimum 6-foot wide, V-shaped, with 6:1 horizontal slope and a minimum longitudinal slope of 0.25%.

j. Engineers may propose other pilot channel material (earthen, stone, etc.) and provide City with proposed horizontal and longitudinal slopes to encourage flow along the bottom of the pond.

k. Detention basins shall be designed such that the 5-year, 10-year, and 100-year, 24-hour storm events are contained within the top of the basin.

DRAFT

l. Where detention ponds outfall to TxDOT infrastructure, pond outfalls shall be sized and configured in concurrence with TxDOT requirements and such that the downstream infrastructure is not adversely impacted.

m. Where detention ponds outfall to any undersized, pond outfalls shall be sized and configured such that the downstream infrastructure is not adversely impacted.

n. Freeboard of 12” from the top of the berm/basin to the 100-year WSE is required.

o. If site conditions prevent the 100-year WSE from being contained within the basin, then the engineer must show that:

   i. $Q_{100,PR}$ is less than or equal to $Q_{100,EX}$.
   ii. Proposed 100-year WSE are lower than Existing 100-year WSE for downstream, upstream, and adjacent properties that are hydraulically connected to the site being developed
   iii. Positive overflow drainage is provided from the basin to the receiving water body, channel, or other drainage infrastructure.
   iv. Flooding or ponding depth within street ROW must be less than 12” deep to account for emergency vehicle access. Storage depth in ROW and lots is measured from the design WSE at the outlet to the proposed finished elevation in the ROW or lot.
   v. Existing surface storage cannot be considered as added storage under proposed conditions. If the site floods during existing conditions, the existing surface storage volume shall be deducted from the total proposed conditions volume.

p. Detention basins shall be situated such that finished floor elevations of adjacent or upstream structures are at least 12” higher than the detained 100-year WSE or 18” higher than the top of embankment of the basin, whichever is higher.

q. Verify that storm drains, pavement and street drainage, channel, and ditches upstream and downstream of the basin function in accordance with City requirements, while taking into account the water levels in the basin and the flows leaving the basin.

r. Berms, structures, and side slope design shall take into account any and all pertinent geotechnical, structural, erosion, and other considerations.

s. For wet detention basins (with normal water levels in the pond, such as golf course irrigation ponds), the storage is calculated only for storage above the normal water surface elevation of the pond. Where a wet pond has an outfall structure, the normal water surface elevation of the pond is calculated as the invert of the lowest outfall structure that is positive flowing and unsubmerged. For outfall structures that are submerged or under static tailwater conditions (submerged outfalls, downstream ponds, below mean sea level, etc.) the normal water surface elevation is calculated as the water surface elevation created by the most downstream outfall. Reference FEMA floodplain maps and data for storm surge and effective floodplain elevations.

DRAFT

t. Provide fences around ponds that are adjacent to public roads or public pedestrian paths.

u. Provide maintenance strips, access ramps, and fences per section 7.9.

Basins are typically located at or near the lowest area of the property. Ponds may also be located higher in the watershed within a property. If so, the design flows must be less than or equal to existing conditions flow at the downstream end of the property.

The use of parking lot detention storage is not recommended, and all other detention options should be fully explored before designing this type of detention facility. It is recommended that preliminary approval be obtained from City Staff prior to beginning a detailed design of any parking lot detention facility. In the event that parking lot storage is used, the maximum ponding depth should not exceed six inches.

Once preliminary detention routing results are obtained, the peak discharge rate from the proposed detention basin shall be compared to the allowable peak discharge rate for each design storm event being analyzed. If the peak discharge rate for any of the design storm events exceeds the allowable peak discharge rate for that event, the size and/or configuration of the outfall structure shall be adjusted until acceptable results are obtained. In addition, the detention basin grading plan may also need to be revised to ensure that adequate volume is provided to attain the appropriate freeboard. If the grading plan is revised, an updated stage-storage relationship will need to be developed and incorporated in the detention routing model.

Figure 7-4. Typical Detention Basin Layout (Plan View)

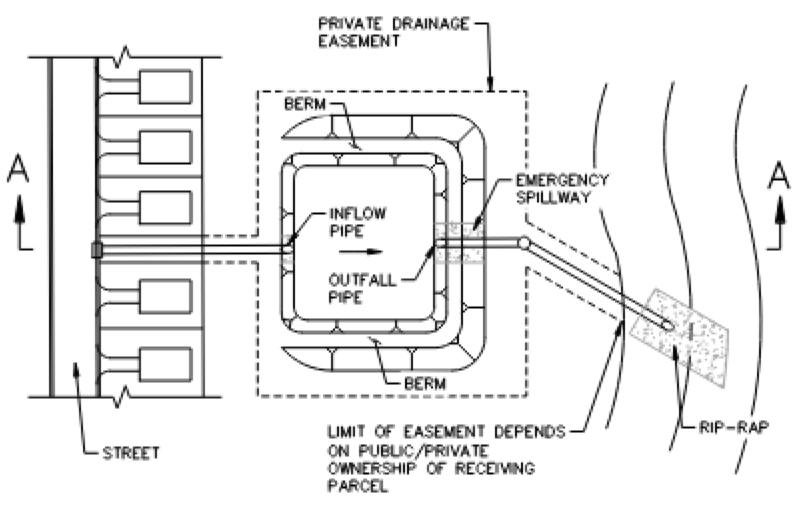
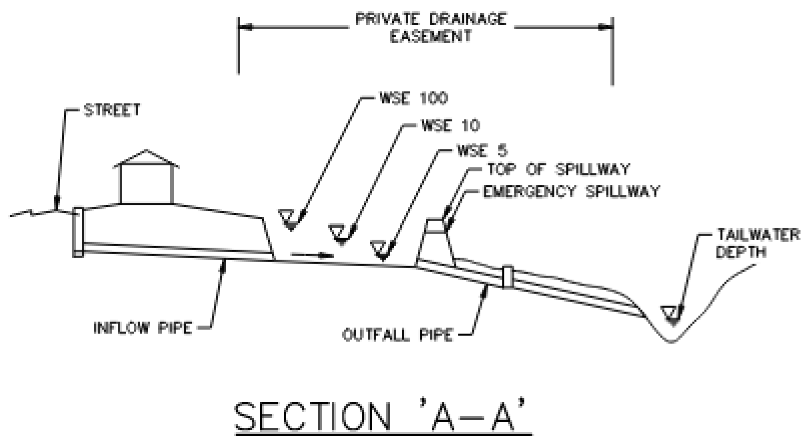


Figure 7-5. Typical Detention Basin Layout (Section View)



7.6 SUBMITTAL REQUIREMENTS

Detention calculation, analysis, and design components to be submitted include but are not limited to:

a. Site Plan, including site layout, pond location, FEMA floodplain, downstream stormwater infrastructure

b. Drainage Area Maps

c. Drainage Plans

d. Grading Plans

e. Detention Plans and Details, including maintenance easements, strips, and ramps

f. Detention Plan Cross-Sections with 5, 10, and 100-year water surface elevations within the pond

g. Outfall structure detail, with 5, 10, and 100-year tailwater elevations

h. Peak Flow Calculations (Existing and Proposed)

i. Inflow hydrographs

j. Stage-Storage Tables

k. Outflow/discharge hydrographs

DRAFT

l. Stage-Discharge Tables

m. Pump calculations, if applicable

n. Hydrologic and hydraulic model files

o. Maintenance plan

Summary tables for peak flow calculations, required detention volume and outflow calculations, stage-storage and state-elevation tables, design water surface elevations, and spillway calculations must be included on the plans. Detailed calculations and supporting information and exhibits must be included in the accompanying drainage engineering report.

After construction, a licensed professional engineer shall certify that the facility is constructed in conformance with design.

7.7 PUMP SYSTEMS

Detention facilities that rely on pumps to discharge all or part of the contributing storm water volume are not recommended. Pumped detention systems will not be maintained by the City of Corpus Christi under any circumstances. Where unavoidable, pumped detention facilities will only be approved under the following conditions.

a. A gravity system is not economically or technically feasible.

b. A duplex pump system, at a minimum, will be provided.

c. The system is designed to accommodate the 100-year peak discharge rate with the largest pump out of service.

d. The pump system outfalls into an existing low, storm drain, ditch, channel, creek, etc.

e. The pump system outfall does not ultimately discharge onto a street or towards a building.

f. The system is designed in accordance with all other storm water system design requirements. For example, discharging pumped runoff from a 100-year storm cannot result in overwhelming a storm drain system designed for a 5-year storm.

g. Fencing of the control panel is provided to prevent unauthorized operation and vandalism.

h. Adequate assurance is provided that the system will be operated and maintained on a continuous basis.

i. An emergency source of power is provided.

j. An extreme event overflow structure is provided so that storm water runoff in excess of the 100-year event is conveyed to the nearest drainage channel without flooding structures. This

DRAFT

overflow structure must also be designed to prevent flooding of structures in the event of pump failure.

7.8 STORMWATER RETENTION

Stormwater retention facilities provide similar benefits as detention facilities by mitigating increased stormwater runoff volume and peak flows. However, retention ponds typically do not have a primary outfall structure and depend on infiltration and evaporation to reduce the water level over time. Basins that do have an outfall structure but have a draw down time greater than 48 hours will be considered retention basins.

Retention ponds will be designed in the same manner as detention ponds, except with zero discharge. The design volume for a retention pond will be double the design volume of a detention pond, to account for two back-to-back 100-year, 24-hour storm events. Retention volume must be contained below the top of the basin and must include an emergency spillway sized to convey the peak flow from one 100-year, 24-hour storm event. A positive overflow pathway from the basin to the receiving stormwater infrastructure is required, similar to detention basins.

7.9 MAINTENANCE CONSIDERATIONS

All detention facilities shall be located in readily accessible areas and at least one access route must be provided to the facility. The addition of chemical pesticides and/or fertilizers is prohibited indetention basins. The following maintenance activities should be performed on a regular basis:

- Mowing;
- Slope repairs;
- Removal of accumulated sediments;
- Removal of trash and debris; and,
- Repairs to discharge structures.

In addition, a maintenance schedule must be prepared in conjunction with the detention design and periodically updated by the agency or entity responsible for maintenance of the detention facility. See the Checklist C-8, which provides an example maintenance plan.

Detention basin maintenance is the responsibility of the owner, POA, or their successor and not the responsibility of the City. The City has the right to enter a stormwater detention facility in the event of emergency.

The owner shall provide a drainage easement that encompasses the detention facility, its maintenance strip, and the outfall structure. Detention basins shall have a minimum 20-foot maintenance strip around the entire detention basin. For ponds with berms, the maintenance strip is measured from the outside toe of the berm.

Detention basins located adjacent to parking lots, private drives, or public roads may reduce maintenance strips to 10’. However, adequate access for maintenance equipment must be provided.

DRAFT

For detention ponds, at least one access ramp must be provided into the bottom of the pond. Ramps must be at least 12’ wide, with a maximum cross slope of 2% and a maximum longitudinal slope of 6:1. If a pond has at least one side slope of maximum 6:1 that is accessible and at least 12’ wide, then a ramp does not have to be included.

Detention ponds may need to be waterproofed to avoid groundwater entering the pond. Owner shall maintain waterproofing at least annually.

For underground detention structures, provide a manhole and riser (if needed) to allow for access into the detention structure. If the detention structure is 400’ or longer, multiple manholes must be provided, spaced no more than 400’ feet apart. Follow all City requirements for manhole dimensions and access considerations.

Where fencing is required, fencing shall be located to deter unauthorized people from entering the pond. Fences and gates shall be placed so that maintenance access is still provided.

DRAFT

Chapter 8
PUMP STATIONS

The purpose of Chapter 8 is to provide information on hydrologic and hydraulic design requirements and criteria for pump stations within the City of Corpus Christi. The Engineer should consult appropriate specialists from other disciplines (i.e., electrical, mechanical, and structural) for assistance with the analysis and design of a pump station. In addition, assistance may be obtained from the following sources: representatives of pump manufacturers, contractors with experience in pump station construction, and representatives of utility firms that will supply power (i.e., electricity, natural gas, and diesel fuel) to the station.

8.1 GENERAL DESIGN REQUIREMENTS

The following design requirements are discussed in this section: purpose of pump stations, pump station components, design storm frequencies, design requirements, and pumped detention facilities.

8.1.1 Purpose of Pump Stations

The purpose of storm water pump stations is to mechanically lift storm water runoff from a lower elevation and discharge it at a higher elevation. Although gravity storm sewer systems are the preferred method of discharging storm water runoff, a pump station may be required if a gravity system is not technically or economically feasible. Pump stations are often used to drain the sag portion of vertical curves on highways and streets or to drain detention basins that are unable to empty by gravity alone. Pump stations can also be used to relieve natural or constructed levee systems such as those in downtown Corpus Christi.

8.1.2 Pump Station Components

A pump station includes the following system components:

a. **Pumps** – Pump selection depends on station layout, required pump rate, wet well depth, and maintenance considerations. Pump selection includes the size, type, and number of pumps. Storm water pump stations generally use vertical propeller and submersible pumps. The system must be designed to accommodate the 100-year peak discharge rate with the largest pump out of service so that breakdown or maintenance of one pump will not interrupt the operation of the pump station. Pump stations must include multiple pumps of varying sizes to facilitate efficient and economical operation during various frequency rainfall events, and to provide backup pumping capacity upon failure of the primary pump.

b. **Motors** – Pump motors are typically 480-volt, three-phase electric motors. However, selection of specific voltage depends on the power available from the utility and the available pump-motor combination. The size of each motor depends on the pump size, flow rate, pressure head, and duty cycle.

DRAFT

c. **Power Sources** – The power source is usually 480-volt, three-phase electrical service provided by the local utility. If available, secondary electrical service feed from a different electrical substation can provide regular power when the primary service power is interrupted. Every pump station shall have an on-site standby electrical generator regardless of the presence of redundant utility power because intense rainfall events often interrupt utility power.

d. **Controls** – Control circuitry includes the water level at which the pump station will be activated, sequence of pump operations, activation and deactivation of the standby generator, and operation of security lighting. Controls may also include automated communication with a central office regarding the pump station water levels, pump readiness, utility electrical power, standby generator fuel level, security, or other concerns.

e. **Structures** – The structure housing a pump station shall meet requirements for public safety, local extreme weather conditions, site security, and maintenance operations. Aesthetics and the possible need for future expansion should also be considered.

f. **Wet Well Sumps** – The wet well sump provides storage for storm water entering the system, which serves to attenuate the hydrograph peak. Since storm water runoff is pumped from the wet well sump, the required pump capacity decreases with increasing wet well storage volume. Therefore, wet well sump dimensions and pump capacities should be determined through trial and error iteration to provide an economical system.

g. **Trash Racks and Grit Chambers** – Wet well sumps shall be designed with provisions for screening trash and other debris associated with the storm water. Convenient access shall be provided for the removal of accumulated debris and silt.

h. **Discharge Conduits** – Pump stations typically discharge into a storm sewer system, an open channel, or a roadside ditch. The design discharge rate shall be based on the suitability of the receiving location, tailwater conditions in the receiving system, and any detention requirements or maximum allowable discharge rates for the area served by the pump station (see Chapter 5). Storm water discharge permits for the receiving system should also be considered.

i. **Security and Access** – Pump stations shall be protected with gates, locks, and fences that provide adequate access to service and maintenance vehicles.

8.1.3 Design Storm Frequencies

The following design storm frequencies shall be used for pump station analysis and design:

a. Pump stations shall be designed using a 100-year design storm event.

DRAFT

b. For pumped detention facilities, analysis of the 5-year and 25-year design storm event is also required to ensure that the detention facility functions properly.

8.1.4 Design Requirements

The following requirements must be met for the design of pump stations:

a. The pump station must be designed for the storm frequencies discussed in Section 7.2.

b. The pump station must be designed to avoid downstream impacts (i.e., a system designed to discharge at the maximum allowable 100-year rate each time the pumps comes on-line could aggravate flooding for more frequent rainfall events).

c. For pumped detention facilities, the pump station analysis shall be performed in conjunction with the detention basin analysis (see Chapter 7).

d. The system must be designed to accommodate the 100-year peak discharge rate with the largest pump out of service.

   An extreme event overflow structure must be provided so that storm water runoff in excess of the 100-year event is conveyed to the nearest drainage channel without flooding structures. This overflow structure must also be designed to prevent flooding of structures in the event of pump failure.

e. An emergency source of power must be provided as described in Section 8.1.2.

f. Fencing of the control panel must be provided to prevent unauthorized operation and vandalism (see Section 8.1.2).

g. Adequate assurance must be provided that the system will be operated and maintained on a continuous basis.

h. Pump station and detention analyses shall be performed concurrently to ensure that the overall detention system is in compliance with applicable criteria and regulations.

8.1.5 Pumped Detention Facilities

Pumped detention facilities are not recommended and the City of Corpus Christi will not be responsible for maintaining any such facilities. In the event that a pumped detention facility is required, it is recommended that the conceptual design be approved by City Staff prior to beginning final design. In addition, the pump station and detention analyses shall be performed concurrently to ensure that the overall detention system is in compliance with applicable criteria and regulations.

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8.2 PUMP STATION DESIGN

The hydraulic design procedure for pump stations is discussed in Sections 8.2.1 through 8.2.9.

8.2.1 Peak Runoff Rates

a. Peak flow rates shall be calculated in accordance with Chapter 4, Hydrology, based on the drainage area size.

8.2.2 Inflow Hydrographs

The following methods shall be used to develop an inflow hydrograph for the drainage area contributing runoff to the pump station.

a. In most cases, a synthetic hydrograph is adequate for the design of pump stations serving up to 200 acres.

b. If project-specific considerations warrant the use of a computed runoff hydrograph, it should be developed using a unit hydrograph method described in Chapter 4 for drainage areas greater than 500 acres.

As shown in Figure 8-1(a), to simplify pump station design for drainage areas up to 200 acres it is assumed that the rainfall occurs at a constant intensity for a certain storm duration. The inflow rate into the storage area is assumed to increase linearly from an initial value of zero at the beginning of the rainfall event until it reaches the peak runoff rate at a time equal to the time of concentration (T_c). For rainfall events with a duration equal to the T_c, the rain ceases at a time equal to the T_c and the runoff rate decreases linearly from the maximum rate to zero in a time period equal to T_c (Figure 8-1(b)). If the storm duration is longer than T_c, the maximum rate of runoff continues at a constant rate until the storm ceases (Figure 8-1(c)).

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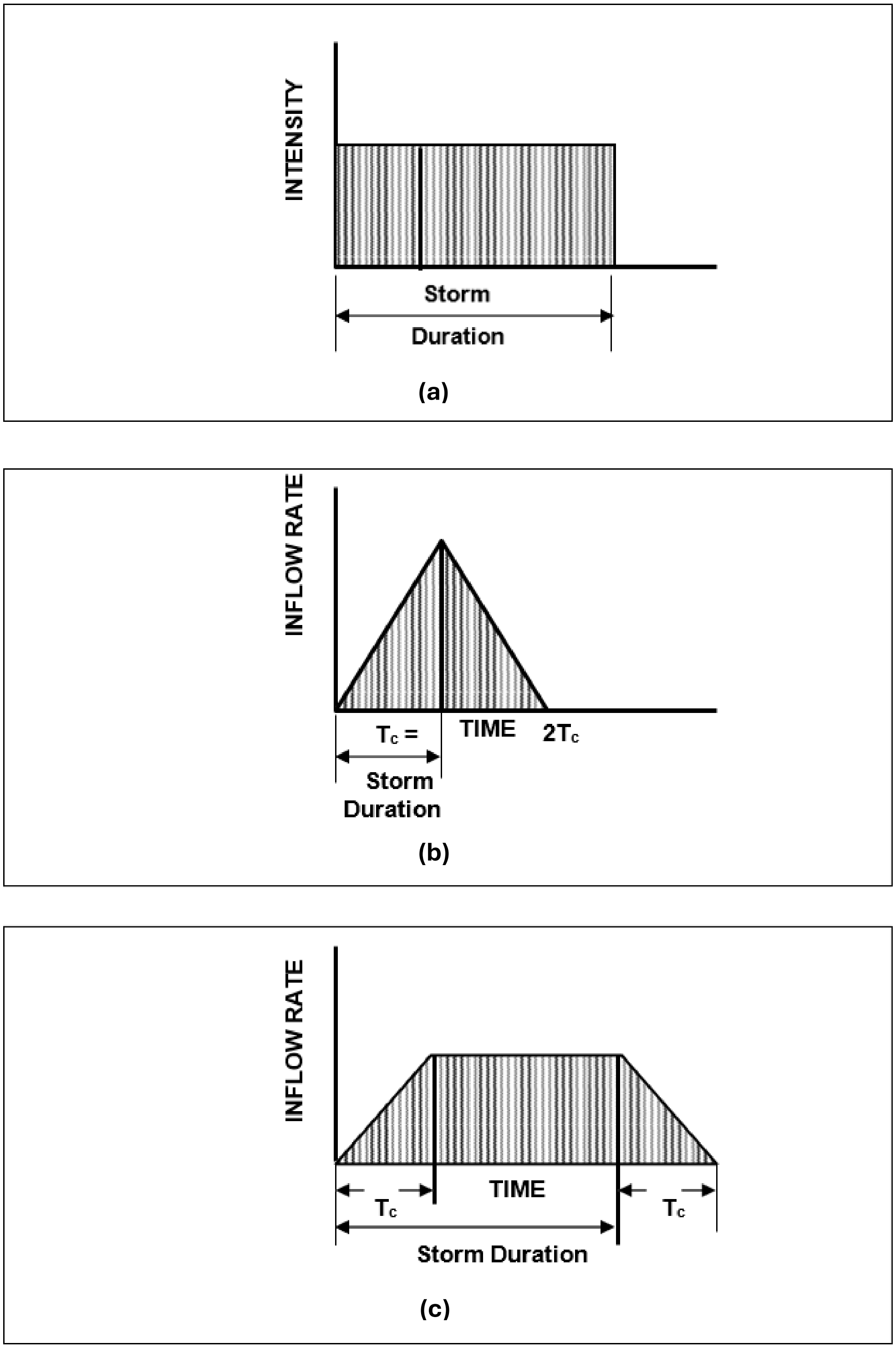
Figure 8-1 (a-c). Synthetic Inflow Hydrographs



8.2.3 Total Runoff Volume

The area under the inflow hydrograph represents the total runoff volume from the drainage area served by the pump station. The following methods shall be used to compute the total runoff volume.

a. As shown in Equation 8-1, the total runoff volume for drainage areas up to 200 acres is calculated as the area under the synthetic inflow hydrograph (Figures 8-2 and 8-3).

$$V = QD$$ **Equation 8-1**

where: V = total runoff volume (cubic feet);
Q = peak runoff rate (cubic feet per second); and
D = storm duration (seconds).

b. The total runoff volume from drainage areas greater than 200 acres shall be computed with the NRCS method or other unit hydrograph method in accordance with the hydrologic methodology discussed in Chapter 4.

8.2.4 Available Storage

The available storage (S) represents storage below the allowable high-water elevation and above the pump cut-off elevation, which is the elevation at which the last pump turns off. Storage volume can be provided in a number of locations, including: wet well sumps, storm sewer systems, detention basins, roadside ditches and streets.

8.2.5 Average Pump Capacity

A preliminary estimate of the average pump capacity (APC) required to remove the total runoff volume is given by Equation 8-2***.***

$$APC = \frac{V_e}{D} = \frac{V-S}{D}$$ **Equation 8-2**

where: AP = average pump capacity (cubic feet per second);
V_e = excess volume (cubic feet);
S = available storage (cubic feet); and
D = storm duration (seconds).

For systems with minimal storage and peak flow attenuation, an APC equal to the peak runoff rate calculated using the procedures described in Section 8.2.1 and 8.2.2 shall be used.

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8.2.6 Stage-Storage Relationship

A stage-storage relationship, which is used to set pump on-off elevations, shall be developed from the available storage at various elevations.

8.2.7 Pump Size and Operation

Based on the APC calculated in Section 8.2.5, an appropriate combination of pumps shall be selected to facilitate efficient operation of the pump station during various frequency rainfall events. The optimum combination typically includes a small sump pump that cycles on and off to empty the wet well sump during frequent rainfall events as well as larger capacity pumps that are reserved for less frequent rainfall events. The starting time of the pumps can be varied as needed to keep the maximum water surface elevation below the allowable high-water elevation during the 100-year design storm event. In addition, it is important to offset pump initiation times by at least one minute to avoid a power overload. The on-off elevations for the pumps must be carefully set to avoid discharging at the maximum allowable 100-year rate each time the pumps come on-line, which could cause downstream impacts during more frequent rainfall events.

The performance of pumped detention facilities shall be analyzed using a 25-year design storm event in addition to the 100-year event. Based on the specific detention facility, a five-year analysis may also be required. In the event that the performance of the pump station is not satisfactory, or the maximum allowable discharge rate is exceeded for either of these rainfall events, the hydraulic design procedures should be repeated until satisfactory results are obtained for all of the design storm events being analyzed. The following system modifications may be required: increasing the available storage, modifying the wet well sump capacity, increasing or decreasing pump capacities, and adjusting pump on-off elevations and combinations.

8.2.8 Total Dynamic Head

The total dynamic head requirement for the pump, including losses and a safety factor can be calculated with Equation 8-3.

$$H_{TD} = h_s + h_f + h_v + \sum h_p + h_{sf}$$ **Equation 8-3**

where:
- H_{TD} = total dynamic head (feet);
- h_s = static head, height to which water must be raised (feet);
- h_f = friction loss in the discharge line (feet);
- h_v = velocity head (feet);
- $\sum h_p$ = summation of friction losses due to pump valves, fittings, and other appurtenances (feet); and
- h_{sf} = safety factor (feet), generally one-foot is adequate to account for *potential* silting or other unpredictable losses.

DRAFT

For assistance in calculating $\sum h_p$, refer to pump appurtenances manufacturers' literature and the minor loss section of Chapter 5. The friction loss in the discharge line (h_f) is given by Equation 8-4. Equation 5-11 can also be used for circular discharge lines.

$$h_f = L\left[\frac{Qn}{0.4644AR^{2/3}}\right]^2$$ **Equation 8-4**

where:
- h_f = friction loss (feet);
- L = length of discharge line (feet);
- Q = peak discharge rate (cubic feet per second);
- *n* = Manning's roughness coefficient, from Table 5-6;
- A = cross-sectional area of flow, assuming full flow in the pipe (square feet); and
- R = hydraulic radius, cross-sectional area divided by wetted perimeter (feet).

The velocity head (h_v) can be calculated from Equation 8-5.

$$h_v = \frac{v^2}{2g}$$ **Equation 8-5**

where:
- h_v = velocity head (feet);
- v = velocity of design discharge in discharge conduit operating at full flow (feet per second); and
- g = acceleration due to gravity (32.2 feet per square second).

8.2.9 Pump Power Requirements

The required horsepower of the pump drivers can be calculated from Equation 8-6. The efficiency can be estimated from potential pump curves or pump manufacturers' literature.

$$P = \frac{\gamma Q H_{TD}}{550E}$$ **Equation 8-6**

where:
- P = required power (horsepower);
- γ = unit weight of water (62.4 pounds per cubic feet); and
- E = anticipated efficiency of motor.

DRAFT

Chapter 9
WATER QUALITY

Additional water quality criteria covered in this *Drainage Design Manual (DDM)* include Erosion and Sediment Control (Chapter 10) and Best Management Practices Chapter 11.

9.1 INTRODUCTION

The Texas Gulf Coast is one of the most biologically rich and ecologically diverse regions of the state. Significant aspects of the City of Corpus Christ's economy depend upon the health and beauty of the surrounding water bodies. A healthy environment is one in which the water quality supports a rich and varied community of organisms and protects public health. Water quality in a body of water influences the way in which communities use the water for activities such as drinking, swimming, or commercial purposes. More specifically, the water may be used by the community for:

- supplying drinking water
- recreation (swimming, boating, nature tourism)
- irrigating crops and watering stock
- industrial processes
- navigation and shipping
- production of edible fish, shellfish and crustaceans
- protection of aquatic ecosystems
- wildlife habitats
- scientific study and education

Since human activities, such as land development, typically have a cumulative negative impact on the quality of the storm water entering the receiving waters, Chapter 9, emphasizes water quality criteria associated with land development.

9.2 WATER QUALITY PROTECTION

Water resources have environmental, social and economic value, and if water quality becomes degraded this resource will lose its value. If water quality is not maintained, it is not just the environment that will suffer - the commercial and recreational value of our water resources will also diminish. Federal, state and local regulations are becoming increasingly strict to ensure the sustainable value of our water resources.

9.2.1 Protect Community Health and Safety

The presence of contaminants and the characteristics of water are used to indicate its quality. These water quality indicators can be categorized as:

- **Biological:** bacteria, algae

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- **Physical:** temperature, turbidity and clarity, color, salinity, suspended solids, dissolved solids
- **Chemical:** pH, dissolved oxygen, biological oxygen demand, nutrients (including nitrogen and phosphorus), organic and inorganic compounds (including toxicants)
- **Aesthetic:** odors, taints, color, floating matter
- **Radioactive:** alpha, beta and gamma radiation emitters

Measurements of these indicators can be used to determine and monitor changes in water quality, and determine whether the quality of the water is suitable for the health of the natural environment and the uses for which the water is required.

9.2.1.1 Environmentally Sensitive Areas

Environmentally sensitive areas are susceptible to being impacted by the effects of storm water runoff from site development. These areas generally contain a multitude of plant, animal, insect, bird, and fish species that depend on good water quality to maintain a healthy ecosystem. All of the natural water bodies in or adjacent to the City of Corpus Christi are considered by the City as environmentally sensitive receiving waters. These natural receiving waters are:[12]

- Corpus Christi Bay
- Gulf of Mexico
- Inner Harbor
- Laguna Madre
- Nueces Bay
- Nueces River
- Oso Bay
- Oso Creek
- Petronilla Creek
- West Oso Creek

Development within or upstream from environmentally sensitive areas

a. Vegetated buffer strips are recommended along boundaries of environmentally sensitive areas, and native vegetation should be utilized where practical.

b. Drainage outfalls that will discharge directly into an environmentally sensitive area shall be located with consideration of natural topography and drainage patterns of the environmentally sensitive area.

c. Velocity control must be provided at outfall openings to eliminate erosion of the environmentally sensitive area, if velocities are high enough to result in scour. For example, rock riprap may be placed at the outfall to allow for velocity reduction.[9]

d. Best Management Practices (BMPs) may be provided to collect sediments and reduce deposition in the receiving waters. Acceptable examples include Grate Inlet Inserts and Detention Basins (see Section 11.4.4 for various structural BMPs that remove sediment).

DRAFT

e. BMPs may be provided to collect floatable debris and prevent floatables from entering the receiving waters. Section 11.4.4 includes examples of various types of Litter Traps that remove floatable debris.

f. BMPs may be provided to collect oils and hydrocarbons; at least the first flush (½-inch) of runoff from the drainage areas shall be collected. Some acceptable BMPs include Sand Filters, Oil/Grit Separators, and Grate Inlet Inserts (see Section 11.4.4).

9.2.1.2 Texas Water Quality Inventory

The Texas Commission on Environmental Quality (TCEQ) regularly monitors the condition of the state's surface waters, and assesses the status of water quality every two years. The TCEQ submits this assessment to the U.S. Environmental Protection Agency (EPA). The report is also published on the TCEQ web-site as the Texas Water Quality Inventory and 303(d) List. The document has essentially two main parts: the Inventory, which gives the status of all the waters in the state, and the 303(d) List, which identifies waters that do not meet one or more of the standards set for their use.

Water quality standards are the basis for regulatory and non-regulatory control of pollutants when the levels of treatment used by permitted dischargers are inadequate to maintain water quality. Some standards are applied generally to many different water bodies, while some are site-specific. Any one water body will usually have multiple uses designated for it. For example, a lake or stream may be designated for use as a source of water for a drinking water treatment plant, for recreation such as swimming and fishing, and as a healthy environment for fish and other aquatic creatures.

After a water body is listed in Category 5, which is the 303(d) list and means the water body does not meet applicable water quality standards or is threatened for one or more designated uses by one or more pollutants, several different courses may be pursued to bring it into compliance with the standards for its use. Further evaluation may be necessary to determine if the current standard is appropriate, or to determine the cause of the impairment. The TCEQ may begin a project to reduce pollution and restore the impaired use under its Total Maximum Daily Load (TMDL) Program. Certain new requirements may also apply for facilities that discharge wastewater into the listed water body. Importantly, the TCEQ may not allow any new or expanded discharges of a listed pollutant into a Category 5 water body if it contributes to the impairment.

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9.2.2 Drinking Water Supply Protection – Environmentally Sensitive Areas

The City's *Storm Water Quality Management: Guidance Document For Developmental Planning & Construction Activities* for the "*Area Adjacent to the Nueces River Water Supply*" indicates that land that is within 1,500 feet from the edge of the Nueces River or any of its tributaries above the Calallen Saltwater Intrusion Dam are designated as environmentally sensitive areas.

Drainage systems upstream from the City's raw water supply intake at the O.N. Stevens water treatment plant must be designed to reduce runoff of contaminated storm water to the water supply.[9] The sub-basins within the Nueces River Basin that outfall into the Nueces River upstream from the O.N. Stevens Water Treatment Plant, have been indicated on the Basin Index – NRB of the Storm Water Master Plan Map Sheets.

The criteria and requirements for protecting the water supply are the same as those required for environmentally sensitive areas, and can be found in Section 9.2.1. of this document. Outfall structures and contributing collection systems within these sub-basins will be required to meet the special environmental standards.

9.2.3 Maintain Ecosystems and Preserve the Natural Environment

An ecosystem is a community of organisms - plants, animals, fungi, and bacteria - interacting with one another and with the environment in which they live. Protecting aquatic ecosystems is in many ways as important as maintaining water quality, for the following reasons:

- Aquatic ecosystems are an integral part of our environment. They need to be maintained if the environment is to continue to support people. World conservation strategies stress the importance of maintaining healthy ecosystems and genetic diversity.

- Aquatic ecosystems play an important role in maintaining water quality and are a valuable indicator of water quality and the suitability of the water for other uses.

- Aquatic ecosystems are valuable resources. Aquatic life is a significant source of protein for humans. In most coastal communities, including Corpus Christi, commercial and sport fishing is economically important.

9.2.4 Regulatory Compliance and Project Requirements

9.2.4.1 Municipal Separate Storm Sewer System (MS4) Permits

The storm water permit program is designed to manage the quality of storm water discharges into the nation's waters. MS4 permits regulate discharges from municipalities and must include a requirement to effectively prohibit non-storm water discharges into storm sewers, and controls to reduce the discharge of

DRAFT

pollutants. Information on the City of Corpus Christi's MS4 permit can be found in the *Storm Water Quality Management* document. A copy of the City's NPDES/TPDES MS4 permit can be obtained from the City's Storm Water Department or viewed on the City's website. The NPDES MS4 permit also includes the responsibilities of the local MS4 Co-Permittees.

9.2.4.2 Storm Water Pollution Prevention Plan

Storm Water Pollution Prevention Plans (SWPPP) shall be developed and implemented for all project sites of 1 acre or more to comply with the TCEQ TPDES Construction General Permit TXR1500000 (see the TCEQ web-site for state permit requirements). These plans will be used during construction and should focus primarily on the prevention of erosion and sediment transport into storm water collection systems (see Checklists C-2 and C-4). When applicable, the SWPPP shall be included in the plan drawings submission process. Storm Water pollution prevention plans should be simple, easy to implement, and easy to maintain through the life of the construction project.

The design components of a SWPPP include the following:

a. Erosion and Sediment Controls

- Soil stabilization practices
- Structural controls and practices
- Storm water management
- Sequence of Construction
- General
- Dust (Aeolian Soils) Control Measures

b. Site Description

- Project limits
- Project description
- Major soil disturbing activities
- Total project area (acres)
- Total area to be disturbed (acres)
- Weighted – runoff – coefficient (after Construction)
- Existing condition of soil and percent vegetative cover
- Name of receiving waters

c. Other Erosion and Sediment Controls

- Maintenance
- Inspection
- Waste Materials
- Hazardous waste
- Sanitary waste
- Offsite vehicle tracking
- Remarks

Rock filter dams are effective measures for preventing sediments from being carried into a creek or channel. The rock filter dam reduces flow velocities, causing suspended sediments to settle out. Sediments accumulating in the area immediately upstream of the rock filter dam must be removed periodically in order to preserve the effectiveness of the dam and the hydraulic capacity of the

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ditch. Filter fabric fences are another effective measure for containing sediments.

Chapter 5 of the Storm Water Quality Management (SWQM) guidance document, published by the City of Corpus Christi, discusses storm water pollution prevention plans in detail. Chapter 4 of the SWQM, which covers best management practices, also provides useful information on managing overland flow and trapping sediment.[12]

9.2.4.3 Pollution Control Plan

Development of sites less than 1 acre require a site-specific pollution control plan. These plans will be used during construction and focus primarily on the prevention of erosion and sediment transport into storm water collection systems (see Checklists C-3 and C-4). When applicable, the Pollution Control Plan shall be included in the plan drawings submission process. The site plan should contain the following:

a. Outline of the site;

b. Delineation of disturbed areas by construction activities;

c. Existing and proposed storm water drainage directional flow lines;

d. Existing and proposed drainage structures;

e. Description of how run-on storm water will be handled, including sheet flow entering the site from adjoining property.

f. Description and location of any environmentally sensitive areas located on the site or adjoining the site, which will receive storm water directly form the site; and,

g. Boundary line between any adjoining State submerged land and the site.

Preliminary boundaries may be used in the preliminary plat; however, the Pollution Control Plan must be amended prior to the final plat.

9.2.4.4 Monitoring Activities (During Construction)

Erosion during the construction phase of both public and private projects is a major cause of siltation of drainage channels and storm sewer conduits. The eroded soil not only clogs the drainage system and reduces its capacity, but also transports organic debris and chemical nutrients to the receiving waters. This leads to increased biological activity and reduced water quality.

DRAFT

The City has adopted a construction guidance manual (*Storm Water Quality Management: Guidance Document for Developmental Planning and Construction Activities*) that includes criteria and technical guidance for development projects from the planning stage through the post-construction stage. The document also addresses water quality concerns after construction, and incorporates special requirements for development that may impact environmentally sensitive areas, such as wetlands and coastal zones.

City Ordinance #022941 requires regular inspections of the effectiveness of the pollution control measures and specifies that good records of those inspections be kept. For construction sites over 1 acre in size, erosion and sediment control measures shall be checked by the Site Manager within 24 hours following a rainfall of 0.5 inches or greater. Additionally, a routine weekly inspection shall be performed by the Site Manager, and an inspection report shall note any damage or deficiencies in the control measures (see Checklist C-5 in Appendix C). The Site Manager shall correct damage or deficiencies, and any changes that may be required to correct deficiencies in the SWPPP shall be made as soon as possible after the inspection. Inspection records shall be maintained by the responsible party.

9.3 QUALITY OF LIFE

9.3.1 Transform Existing Facilities into Neighborhood Assets

Existing channels and detention facilities can be beautified by acquiring adjacent land to the structures to serve as open space, buffer zones, and greenways. With the cooperation of the Parks and Recreation department, the City will coordinate and promote the creation of multi-use facilities and greenway inter-connected park areas. An example of a multi-use facility is a storm water detention basin that can be used by area residents as soccer fields during dry interludes. See Chapter 7 for detention criteria.

Another example is the combination of a storm water detention system with a park and a civic center. In addition to walking, jogging and cycling trails, the park could feature a wetlands basin created with islands and a connected boardwalk. The area would provide year-round sanctuary for local and migratory birds, and, in case of extreme storm water events, the wetlands would serve as a detention basin. Visitors would be encouraged to enjoy bird watching, walking and picnicking.

9.3.2 Establish Greenway Corridors

A greenway is a corridor of undeveloped land in or near a city that is designed for recreational use. Properly designed greenways can be combined with drainage facilities and provide residents with bikeways and walkways as alternative modes of transportation, nature trails, park and recreation areas, and flood protection.

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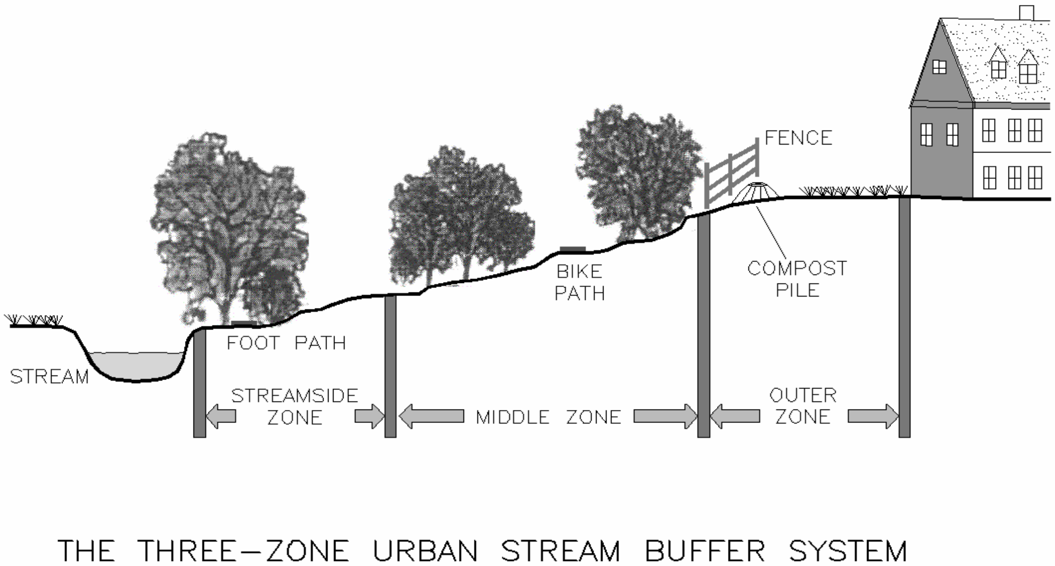
Refer to the City's Urban Transportation Plan and Parks and Recreation Master Plan for greenway corridors and associated planning and development requirements.

Figure 9-1 shows a typical cross-section of an urban stream with a vegetated buffer. Bike and walking paths have been incorporated into the design.

Figure 9-1. Example of a Greenway



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The following guidelines shall be considered when designing a greenway. See also the City of Corpus Christi's Unified Development Code.

a. Utilize riverbank stabilization strategies, such as bioengineering (see Chapter 10), that enhance the river and riverbank ecosystems.

b. Where appropriate, integrate public access to the water that is safe and supportive of adjacent riverbank areas.

c. Integrate a variety of vegetation above and below ordinary high water, which supports the river and riverbank habitats.

d. Ensure that pedestrian and bicycle connections to the greenway trail from the adjacent access ways or urban spaces are safe, convenient and direct.

e. Ensure that the greenway trail, its access connections, and the access ways are well lit at night to create a sense of activity and security. Place and shield lighting fixtures so that they do not detract from adjacent use areas.

f. Align the trail to take advantage of the site's opportunities to enhance the diversity of trail experiences. Special topographical features, unique views, special access to the river/bay, and new emphasis areas are examples of opportunities that could cause the trail to wander.

g. Define viewpoints that are understood as extensions of the greenway trail, without conflicting with the trail's movement functions. They should provide enough space for groups of people to gather without conflicting with the movement portions of the trail system.

h. Consider the incorporation of "short-duration stop" facilities in viewpoint design. Short- duration stop facilities support stopping, gathering, and viewing activities. Some examples include places to sit, interpretive kiosks, integrated water features, public art, and access to the water.

i. Select appropriate species of native and native-like plants based on the soil, light, moisture conditions, context, and adjacent uses of the site.

j. Arrange plant communities to provide ecological functions, security, and connectivity to urban spaces.

Table 9-1 below shows potential measures for protecting streams and establishing greenway corridors.

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Table 9-1. Potential Stream Protection Measures

Characteristics	Streamside Zone	Middle Zone	Outer Zone
Function	Protect the physical integrity of the stream ecosystem.	Provide distance between upland development and streamside zones.	Prevent encroachment and filter backyard runoff.
Width	Minimum 25 feet plus wetlands and critical habitats.	50 to 100 feet depending on stream order, slope, and 100 year floodplain.	25 foot minimum setback to structures.
Vegetative Target	Undisturbed Mature forest; reforest if grass.	Managed forest, some clearing allowable.	Forest encouraged, but usually turfgrass.
Allowable Uses	*Very Restricted* flood control, utility right of ways, footpaths, etc.	*Restricted* some recreational uses, some storm water BMPs, bike paths, tree removal by permit	*Unrestricted* residential uses including lawn, garden, compost, yard wastes; most storm water BMPs.

9.4 ENFORCEMENT

City Inspectors are charged with the responsibility to confirm compliance with City Ordinance #022941. A City Inspector may periodically inspect the construction site during construction for compliance with the requirements of City Ordinance #022941. Inspectors from the EPA or TCEQ may inspect the site as well. The Site Manager shall allow Inspectors to:

a. Enter the premises where a regulated facility or activity is located or conducted;

b. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under the NPDES permit; and,

c. Sample or monitor at reasonable times, for the purposes of assuring permit compliance or as otherwise authorized by the Clean Water Act, any substance or parameters at any location.

9.4.1 Inspections

A City Inspector may also inspect the construction site after construction to ensure that the permanent BMPs listed in the Storm Water Management Plan have been implemented. Checklist C-6 may be used during the inspection and all items must be satisfied before an occupancy permit will be issued.

The City may inspect all properties maintained under its jurisdiction. This includes trash and silt removal from ditches, channels, and storm sewers; routine mowing of vegetated

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channels and swales; routine removal of vegetative growth in lined channels; and regular repairs to damaged facilities. Checklist C-7 indicates the frequency of maintenance required on various facilities, and the items or components to be inspected.

9.4.2 Violations of Permit Conditions

The City's NPDES permit enables the City to enforce the conditions of the permit and of the *Clean Water Act (CWA)*. Violators are subject to federal, state, and municipal enforcement measures that can include both monetary penalties and imprisonment (see Section 309 of the CWA for a more detailed description).

When requirements of City Ordinance # 022941 (or subsequent ordinances) are not met, Inspectors will issue a warning to the contractor. The contractor then has twenty-four (24) hours to resolve the problem. If proper corrective action is not taken within twenty-four hours of the receipt of the warning, the Storm Water Department is authorized by the Ordinance to issue a citation. Violations carry fines of up to $500 per day per violation for as long as the violation exists.

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Chapter 10
EROSION AND SEDIMENT CONTROL

10.1 INTRODUCTION

Erosion rates on disturbed land may increase from 100 to 1000 times that of pre-construction rates. According to the U.S. Environmental Protection Agency (EPA), sediment runoff rates from construction sites are typically 10 to 20 times greater than those of agricultural lands and 1000 to 2000 times those of forest lands. Designing drainage systems which efficiently drain an area as quickly as possible is often contrary to reducing erosion or controlling sediment. This section of the manual describes methods for controlling erosion and sediment deposition in drainage facilities in the City of Corpus Christi. Several of the best management practices (BMPs) discussed in Chapter 11 of this document also discuss erosion and sediment control.

Additional information can be found in the Storm Water Quality Management (SWQM) document published by the City of Corpus Christi. Twelve topics covered include Basic Principles of Erosion and Sediment Control and Best Management Practices, which includes information on sediment control.

10.2 EFFECTS OF EROSION AND SEDIMENTATION

Erosion and sedimentation can have very serious effects on *storm water* drainage, and ultimately the quality of the receiving water in which it is discharged. Some of these effects include:

a. Integrity of Drainage Facilities: Erosion can cause slope failures, increase roughness coefficients, and reduce the efficiency of drainage channels. Sediment deposition can clog drainage culverts and reduce the available conveyance in open channels.

b. Maintenance: Erosion can significantly reduce the maintainability of drainage facilities and increase the cost of maintenance by increasing the frequency with which repairs are required.

c. Water Quality: Erosion and sedimentation can increase the turbidity of water, which distresses aquatic life and inhibits growth of the sea grasses that serve as a place of spawning and a nursery for the young. Pollutants such as heavy metals, oil and grease, and fertilizers attach to soil particles compounding the water quality problems caused by the sediment itself.

10.3 AREAS WITH HIGH EROSION POTENTIAL

To protect areas with high erosion potential, an erosion control plan shall be prepared (see Checklist C-4). Areas with relatively high erosion potential include the following:

a. In channel bends, especially where the radius of curvature is less than three times the top width of flow in the channel.

b. Around bridges and culverts, where channel transitions and reduced flow areas create increased flow velocities.

c. In steep sections of channels and ditches and on steep, unprotected slopes where flow velocities may reach erosive levels.

d. Along grass-lined channel side slopes where significant amounts of storm runoff pass over the channel bank and run down the sides of the channel.

e. At confluences where flows in tributary channels, storm sewers, or roadside ditches enter a receiving channel.

f. In areas where non-cohesive soils are particularly prone to erosion.

10.4 SLOPE PROTECTION METHODS

This section reviews some of the more common slope protection methods.

10.4.1 Turf Establishment

The establishment of grasses on exposed earthen side slopes is the most common method for protecting the slopes from erosion. Grass establishment shall be initiated as quickly as possible after channel construction or repair work is completed, but no more than 14 days after the construction activity has ceased. Turf reinforcement mats may be required for geostabilization and to expedite turf establishment.

The grasses used for this purpose shall be hardy, salt tolerant varieties which do not require repeated watering and excessive amounts of care once they are established. Grasses with deep root systems are preferable to those with shallower systems because they are more resistant to drought. The design criteria and requirements for sodding and seeding of grassed waterways can be found in Section 4.4 of the SWQM document.

10.4.2 Slope Paving

Concrete slope paving is an effective slope protection method but is costly to apply over large areas. Concrete slope paving is generally discouraged, but it may be used in limited areas where the potential for erosion is very high, such as channels with limited right-of-way where steep side slopes would likely be susceptible to erosion. See Chapter 5 for slope criteria.

The slope paving shall be maintained and inspected as follows:

a. Lined waterways shall be inspected weekly for 3 months after installation. Afterwards, inspect channels at regular intervals as well as after major rains. Make repairs promptly.

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b. Concrete-lined channels shall be inspected annually to assure there is no undermining.

c. Outlets shall be checked for scour.If there is potential for scour, appropriate energy dissipation measures shall be taken.

d. Carefully check road crossings for indications of bank failures, scour holes, and undermining; make repairs immediately.

Maintenance and inspection, as referenced in a. through d. above, shall mean: a private developer for the first year after construction completion under the project's warranty period and thereafter shall mean the City of Corpus Christi.

10.4.3 Interlocking Blocks

Pre-cast concrete block revetments consist of pre-formed sections which interlock with each other, are attached to each other, or butt together to form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation but share certain common features. These features include flexibility, rapid installation, and provisions for the establishment of vegetation within the revetment. Pre-cast revetments are bound using a variety of techniques. In some cases the individual blocks are bound to rectangular sheets of filter fabric (referred to as fabric carrier). Other manufacturers use a design which interlocks individual blocks, while other units are simply butted together at the site. The interlocking method required by the City of Corpus Christi is to join individual blocks with wire cable or synthetic fiber rope. Use a City-approved, wire-cable or synthetic-fiber-rope tied revetment manufacturer and follow the manufacturer's recommended design procedure.

10.4.4 Rock Riprap Scour Protection

Rock riprap consists of rock or broken concrete in pieces with a minimum dimension of 6 inches. Rock riprap is normally placed as a layer which begins 18 inches below the finished channel grade.

10.4.4.1 Design Criteria

The following criteria shall be considered during design.

a. The minimum mat thickness shall be 18 inches.

b. Well-graded blocks weighing from 40 pounds to 265 pounds should be used.

c. The maximum steepness of slopes protected by rock riprap shall be 2 horizontal to 1 vertical.

DRAFT

d. Filter fabric bedding is required in areas where rock riprap is placed on sandy or silty soils. On cohesive clay soils with very little sand content (less than 20% sand), filter fabric is not required.

e. Sacks of commercially available concrete mix may not be used as rock riprap because the lack of gradation allows water penetration and undermining of the soil under the installation.

10.4.4.2 Construction Criteria

The following criteria shall be considered during construction.

a. The area shall be prepared by first clearing all trees and debris and grading the surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 inches. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions.

b. Any debris found buried near the edges of the revetment shall be removed.

c. The common methods of rock riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the best rock riprap revetment, but it is the most expensive method. Steeper side slopes can be used with hand placed rock riprap than with other placing methods. Where steep slopes are unavoidable (when channel widths are constricted by existing bridge openings or other structures, and when ROWs are costly), hand placement should be considered.

d. In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone and can result in an overly rough revetment surface. Stone should not be dropped from an excessive height as this may result in the same undesirable conditions.

e. Rock riprap placement by dumping with spreading may be satisfactory provided the required layer thickness is achieved. Rock riprap placement by dumping and spreading is the least desirable method as a large amount of segregation and breakage can occur and is not recommended. In some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method.

DRAFT

10.4.4.3 Inspection

Rock riprap shall be maintained and inspected as follows:

a. Rock riprap lined waterways shall be inspected by the contractor weekly for 3 months after installation.

b. Rock riprap shall be checked annually for dislodged stones and to assure that scour does not occur beneath the rock riprap layer. Repairs shall be made immediately.

Maintenance and inspection, as referenced in a. and b. above, shall mean: a private developer for the first year after construction completion under the project's warranty period and for public facilities, thereafter shall be maintained and inspected by the City of Corpus Christi.

10.4.4.4 Filter Fabric

To provide good performance, a properly selected filter fabric (from a City approved manufacturer) shall be installed in accordance with manufacturer recommendations with due regard for the following precautions.

a. Heavy rock riprap may stretch the cloth as it settles, eventually causing bursting of the fabric in tension. A 4-inch to 6-inch gravel bedding layer shall be placed beneath the rock riprap layer for gradations having a median diameter (d_{50}) greater than 3.0 feet.

b. The filter cloth shall not extend into the channel beyond the rock riprap layer; rather, it shall be wrapped around the toe material as illustrated in Figure 10-1.

c. Adequate overlaps, as described in the manufacturer's installation instructions or design engineer's instructions, must be provided between individual fabric sheets.

d. A sufficient number of folds, as described in the manufacturer's installation instructions or design engineer's instructions, shall be included during placement to eliminate tension and stretching under settlement.

e. Securing pins with washers are recommended at 2- to 5-foot intervals along the midpoint of the overlaps.

f. Proper stone placement on the filter requires beginning at the toe and proceeding up the slope. Dropping stone from heights greater than 2 feet can rupture fabrics (greater drop heights are allowable under water).

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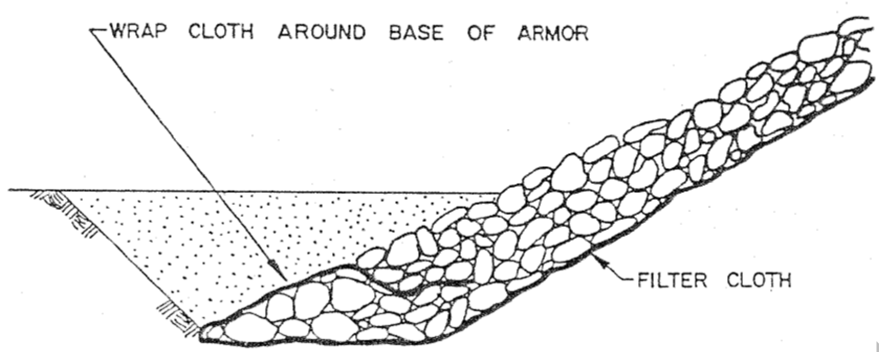
Figure 10-1. Filter Fabric Placement for Rock Riprap



10.4.5 Bioengineering Methods

Bioengineering combines mechanical, biological, and ecological concepts to construct "living" structures for bank and slope protection. Bioengineering methods use structural support to hold live plantings in place while the root structure grows, and the plants are established. This is done through the use of sprigging, live crib walls, cut brush layers, live fascines, live stakes, and other methods.

Advantages of bioengineering include: natural appearance, self-healing properties, habitat enrichment, and resistance to slope failure. Disadvantages include: labor-intensive installation, need for stability control until the roots are established, and dependence on materials to root and grow.

Soil-bioengineered bank stability systems have not been standardized; the decision of whether and how to use them requires careful consideration. Two excellent references for detailed bioengineering design guidelines are *Stream Corridor Restoration: Principles, Processes, and Practices* and *Engineering Field Handbook*, both of which are published by the Natural Resources Conservation Service. Chapter 16 of the *Engineering Field Handbook* discusses Streambank and Shoreline Protection, and Chapter 18 covers Soil Bioengineering for Upland Slope Protection and Erosion Reduction.

These documents provide background on fundamental concepts necessary for planning, designing and applying bio-engineering techniques on many streams. Expertise in soils, biology, plant sciences, landscape architecture, geology, engineering and hydrology may be required for projects where the stream is large, or the erosion is severe. Several examples of bio-engineering techniques are presented in Figures 10-2 through 10-6.

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Figure 10-2. Live Cribwall Details

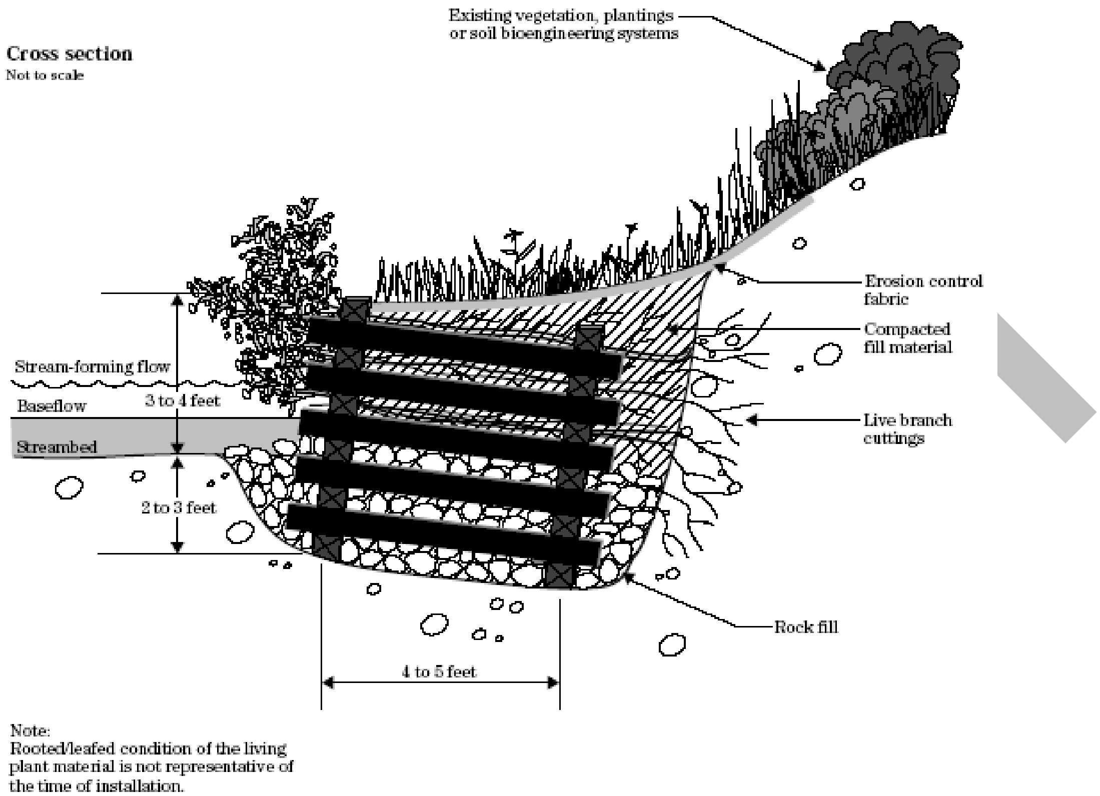

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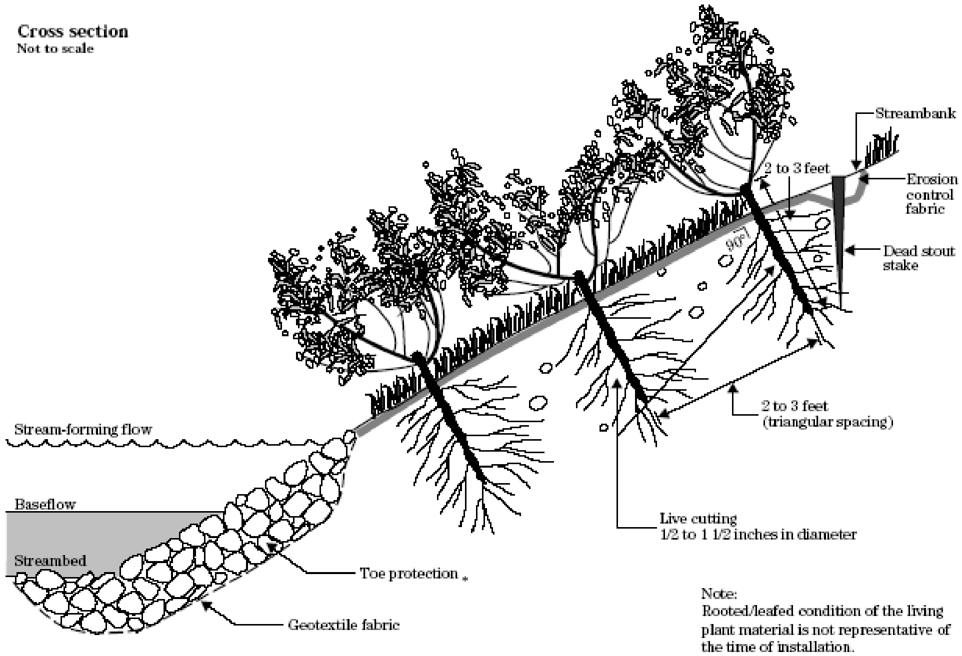
Figure 10-3. Live Stake Details



* Note: The toe of the slope is protected by the mass of rocks on top of the geotextile fabric.

Figure 10-4. Live Fascine Details

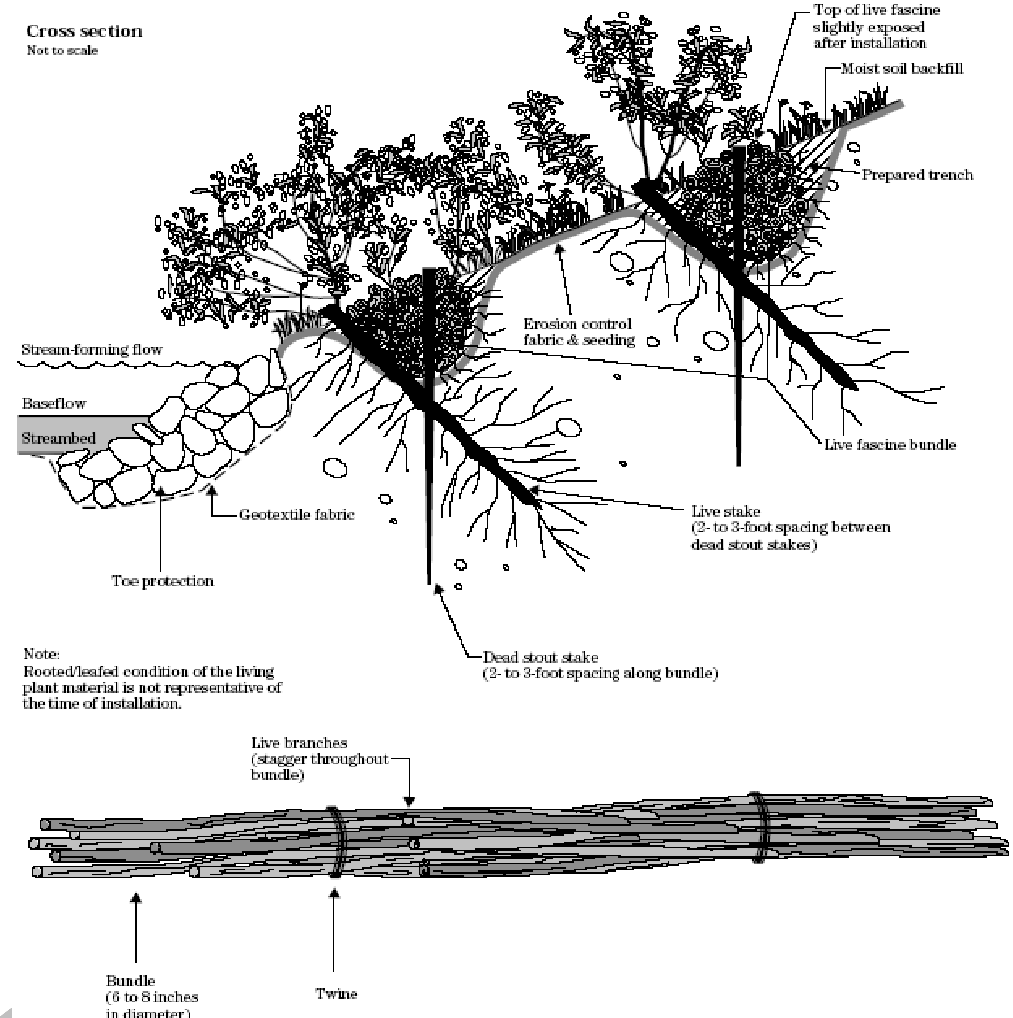


Figure 10-5. Brush Mattress Details

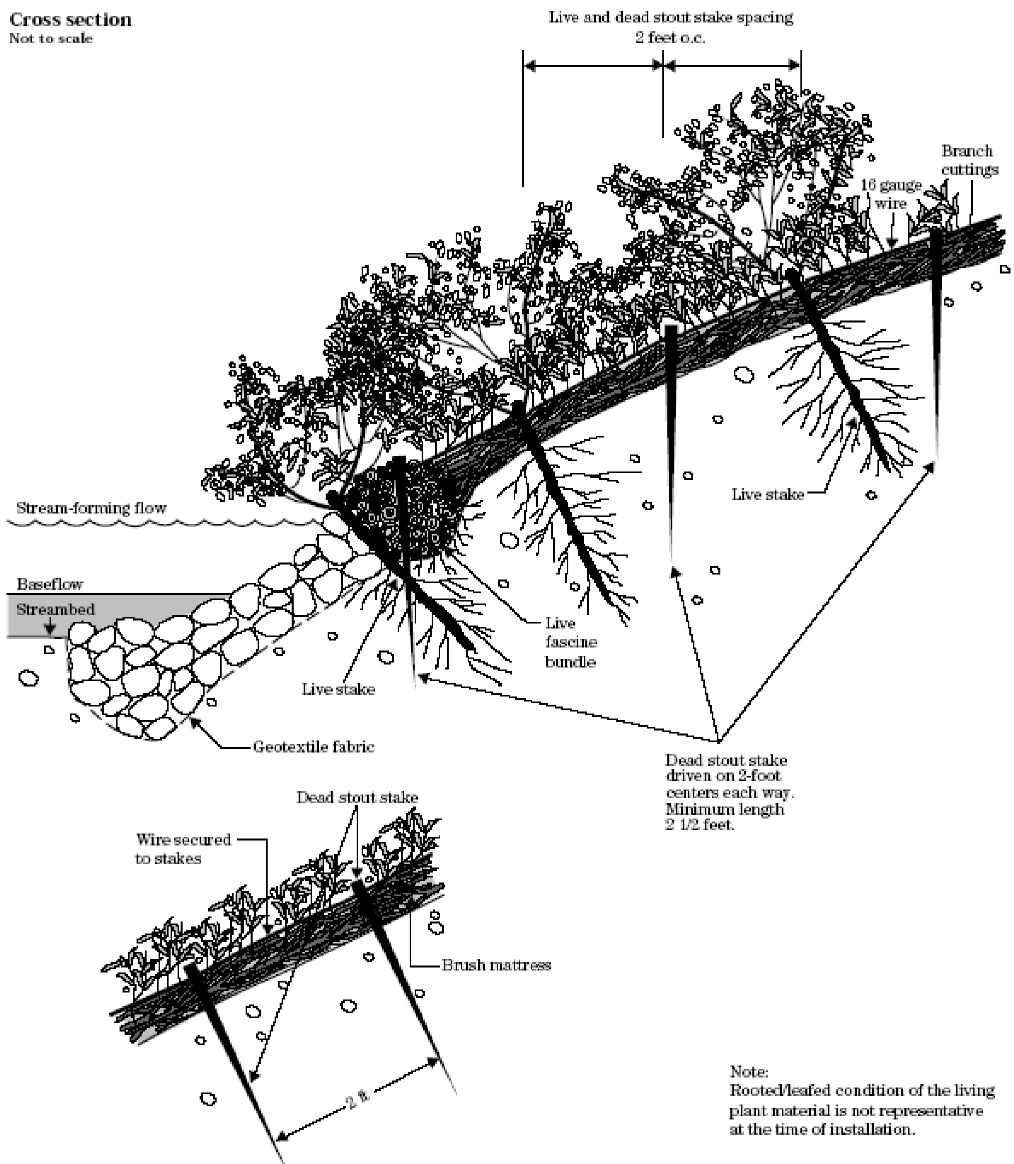
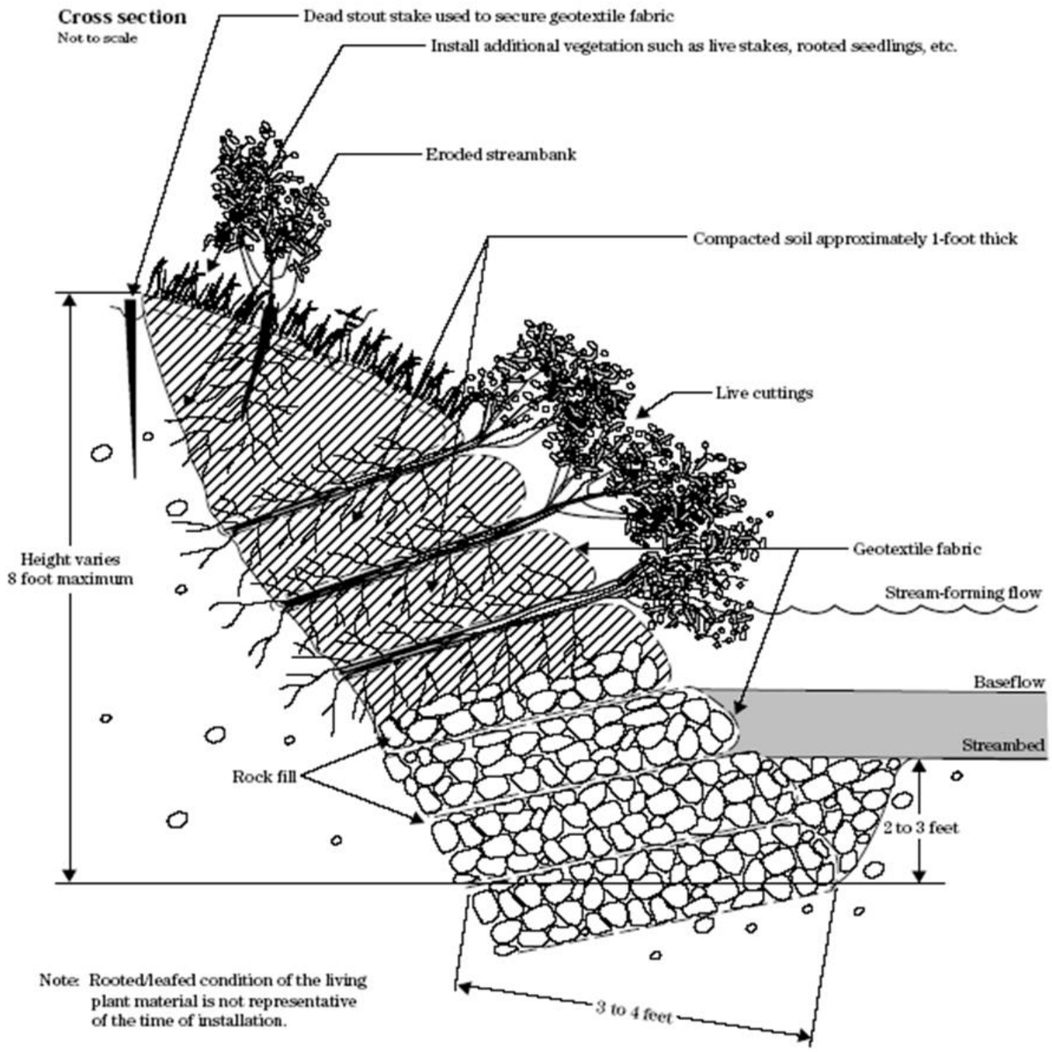


Figure 10-6. Vegetated Geogrid Details



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10.5 REQUIREMENTS FOR CHANNEL BENDS AND CONFLUENCES

Erosion protection is required for all channel bends with a radius of curvature (measured along the channel centerline) less than three times the top width of flow in the channel. When required, erosion protection must extend along the outside bank of the bend and at least 20 feet upstream and downstream of the tangent points. Slope protection on the channel bottom and the inside bank is required only if anticipated flow velocities are above non-erosive levels.

Figure 10-7 illustrates the minimum requirements for erosion protection and channel lining at the confluence of two open channels. Table 10-1 shall be used to determine whether erosion protection is needed given the angle of intersection between the channels and the anticipated velocity in the tributary channel. Table 10-2 summarizes the minimum extent of erosion protection upstream and downstream of the confluence.

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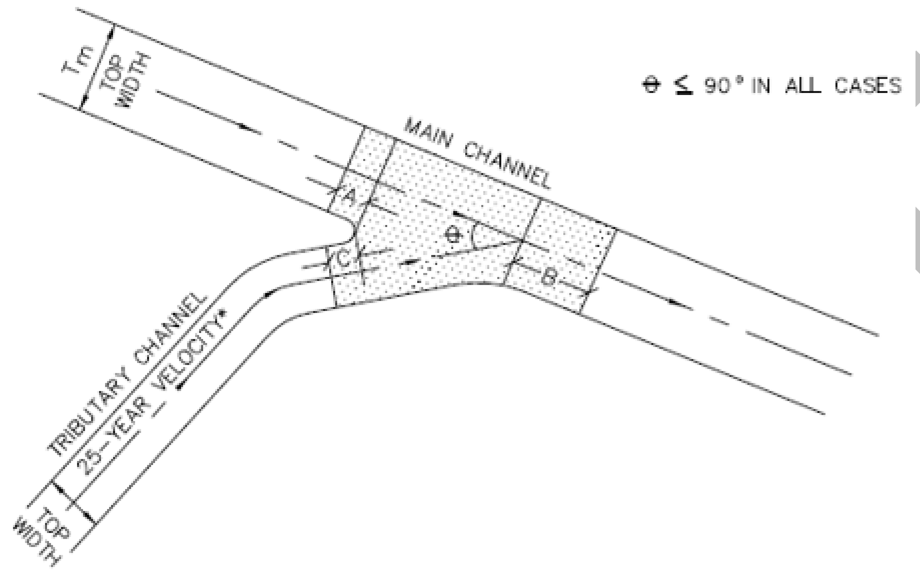
Figure 10-7. Erosion Protection Requirements at Stream Confluences



For both bends and confluences, the top edge of erosion protection shall extend to the channel top of bank. A healthy grass cover must be established on the channel slope and maintenance strip above the concrete lining.

DRAFT

Table 10-1. Minimum Erosion Protection for Channel Confluences

Velocity in Tributary Channel (feet per second)	Angle of Intersection (Θ)	
	15 to 45 degrees	**45 to 90 degrees**
4 or more	Protection Required	Protection Required
2 to 4	No Protection Required	Protection Required
2 or less	No Protection Required	No Protection Required

Table 10-2. Minimum Extent of Erosion Protection at Confluences

Location	Minimum Distance (feet)
A (Main channel, upstream of intersection)	20
B (Main channel, downstream of intersection)	larger of 50 or $0.75T_m/\tan \Theta$
C (Tributary channel, upstream of intersection)	20

*Note: Velocity in tributary channel assuming no backwater from main channel.

10.6 REQUIREMENTS FOR STORM SEWER OUTFALLS

Storm sewer outlets shall be as follows.

a. All outlets shall be designed so that velocities will be appropriate to, and will not damage, receiving waterways.

b. Outlet protection using riprap or other approved materials shall be provided as necessary to prevent erosion.

c. The soils above and around the outlet shall be compacted and stabilized to prevent erosion around the structure.

10.7 CHANNEL BACKSLOPE DRAIN SYSTEMS

Backslope drain systems intercept sheet flow which otherwise would flow over the banks of drainage channels and down the side slopes. The drainage swales can be concrete-lined (Figure 10-8) or grass-lined, with post inlets to intercept the water and route it into the channel. The purpose of backslope drain systems is to prevent erosion, which sheet flow over the banks could cause. The following minimum requirements shall be applied to all backslope drainage systems.

a. The minimum backslope drain pipe diameter shall be 24 inches.

DRAFT

b. The maximum spacing between backslope drains shall be 600 feet.

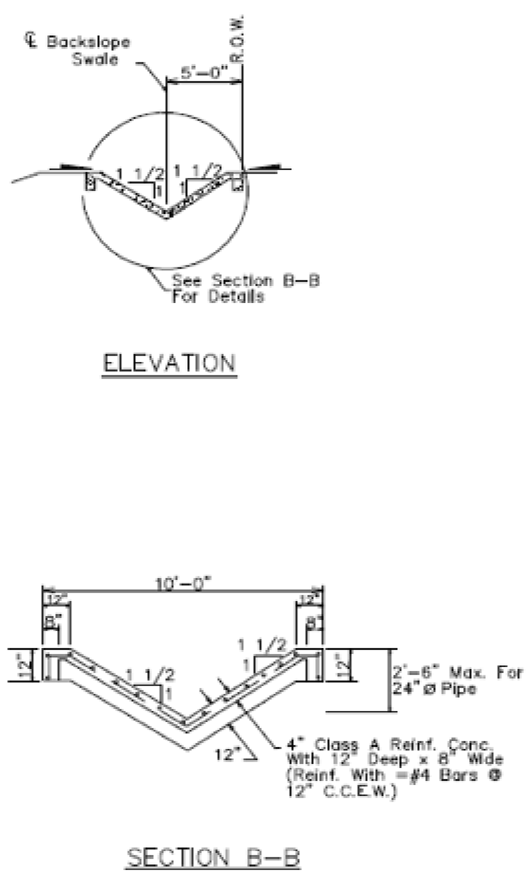
c. The center-line of the backslope drainage swale shall be located five feet inside the channel right-of-way line when 20-foot maintenance strips are used.

d. The minimum depth for backslope drainage swales shall be 0.5 feet. The maximum depth shall be 2.0 feet.

e. The minimum invert slope for backslope drainage swales shall be 0.2%.

f. The maximum side slope for backslope drainage swales shall be 1.5 horizontal to 1 vertical.

Figure 10-8. Typical Concrete-Lined Backslope Interceptor Swale



10.8 PAVED FLUME

The purpose of the paved flume is to convey surface runoff safely down slopes without causing erosion. It may be used temporarily during construction of site improvements (as a Best Management Practice) and/or permanently for final development of the site; the maximum allowable drainage area is 36 acres. Additional information can be found in the *SWQM* document in section 4.1.4 (Paved Flume).

Paved flumes are designated by size groups, which are a function of the following dimensions.

Size Group A:

- Height (H) of the dike at the entrance is at least 1.5 feet.
- Depth (D) of the chute down the slope is at least 8 inches.
- Length (L) of the inlet and outlet sections is 5 feet.

Size Group B:

- Height (H) of the dike at the entrance is at least 2 feet.
- Depth (D) of the chute down the slope is at least 10 inches.
- Length (L) of the inlet and outlet sections is 6 feet.

The size is designated with a letter and number such as A-6, which denotes a Size Group A chute or flume with a six-foot bottom width. The selected size shall be shown on the plans. Each size group has various bottom widths and allowable drainage area as shown in Table 10-3. If a minimum of 75% of the drainage area will have a good grass or woodland cover throughout the life of the structure, the drainage areas listed in Table 10-3 may be increased by 50%. If a minimum of 75% of the drainage area will have a good mulch cover throughout the life of the structure, the drainage areas listed in Table 10-3 may be increased by 25%.

Table 10-3. Paved Flume Size Groups

Size	Bottom Width (b), Feet	Maximum Drainage Area, Acres
A-2	2	5
A-4	4	8
A-6	6	11
A-8	8	14
A-10	10	18
B-4	4	14
B-6	6	20
B-8	8	25
B-10	10	31
B-12	12	36

DRAFT

The following minimum requirements shall be applied to all paved flumes.

a. A temporary paved flume may be constructed of hot mix asphaltic concrete or un- reinforced Portland cement concrete. A permanent paved flume shall have reinforcing steel.

b. Portland cement concrete shall be 5 sack, 3000 psi slope paving concrete.

c. Soil around and under the entrance section shall be tamped to the top of the embankment in lifts appropriately sized for the method of compaction utilized.

d. Subgrade shall be constructed to the required elevations. All soft sections and unsuitable material (as determined by a Registered Professional Geotechnical Engineer) shall be removed and replaced. Compact subgrade thoroughly and shape to a smooth, uniform surface.

e. Fill material for embankment shall be free of roots, woody vegetation, oversized rocks, or organic or other objectionable matters. The area under the embankment shall be cleared, grubbed, and stripped of vegetation and root mat.

f. When a paved flume is used, the velocity at its outfall shall be checked for erosion potential downstream. When required, energy dissipation structures shall be installed.

g. The Owner/Developer shall inspect temporary paved flumes within 24 hours after each rainfall of 0.5 inches or greater; daily during periods of prolonged rainfall; and at least once per week. Repair damaged sections. Redress and replace stone or riprap at the outlet as needed.

h. Remove sediment from the stabilized outlet when sediment has accumulated to a depth of one foot.

10.9 OUTLET STABILIZATION & ENERGY DISSIPATION STRUCTURES

The goal of outlet stabilization is to prevent erosion at the outlet of a channel or conduit by reducing the velocity of flow and dissipating the energy. This practice applies where the discharge velocity of a pipe, box culvert, diversion, open channel, or other water conveyance structure exceeds the permissible velocity of the receiving channel or disposal area. The type of energy dissipator selected for a site must be appropriate to the location.

Energy dissipators shall be employed whenever the velocity of flow leaving a storm water management facility exceeds the velocity that will cause erosion of the downstream channel system. Sections 10.9.1 through 10.9.4 describe several types of energy dissipators; alternate types may also be allowed.

DRAFT

10.9.1 Riprap Aprons

Description

A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. This is the most commonly used practice because of the relatively low cost and ease of installation. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. The riprap apron shall be extended downstream until stable conditions are reached even though this may exceed the length calculated for design velocity control. Riprap aprons are appropriate when the culvert outlet Froude Number (Fr) is less than or equal to 2.5. The Froude Number is a dimensionless ratio of the inertial and gravitational forces:

$$Fr = \frac{V}{\sqrt{gD}}$$ **Equation 10-1**

Where V = average velocity of flow (feet per second);
g = gravitational acceleration (32.2 feet per square second); and
D = hydraulic depth (feet).

Riprap-stilling basins (see Section 10.9.2) or plunge pools reduce flow velocity rapidly. They should be considered in lieu of aprons where overfalls exit at the ends of pipes or where high flows would require excessive apron length.

Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter (d_{50}). If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron shall be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps.

Step 1: If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (d) (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 10-9 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 10-10 should be used.

Step 2: Determine the correct apron length and median riprap diameter (d_{50}), using the appropriate curves from Figures 10-9 and 10-10. Also determine the apron width (W) using the equation in the figure in the upper left corner of Figures 10-9 and 10-14. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 10-11.

DRAFT

a. **For pipes flowing full**: Use the depth of flow (d), which equals the pipe diameter (in inches), and design discharge (in cubic feet per second) to obtain the apron length (L_a) and median riprap diameter (d_{50}) from the curves in the appropriate figure.

b. **For pipes flowing partially full**: Use the depth of flow (d) in inches, and velocity (v) in feet per second. On the lower portion of the appropriate figure, find the intersection of the d and v curves; then find the riprap median diameter (d_{50}) from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth (d). Find the minimum apron length (L_a) from the scale on the left.

c. **For box culverts**: Use the depth of flow (d) in inches, and velocity (v) in feet per second. On the lower portion of the appropriate figure, find the intersection of the d and v curves; then find the riprap median diameter (d_{50}) from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth (d). Find the minimum apron length (L_a) using the scale on the left.

Step 3: If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron shall be as shown in Figure 10-11. This will provide protection under either of the tailwater conditions.

<u>Design Guidelines</u>

The following items shall be considered during riprap apron design:

a. The maximum stone diameter shall be 1.5 times the median riprap diameter (d_{max} = $1.5d_{50}$, d_{50} = the median stone size in a well-graded riprap apron).

b. The riprap thickness shall be 1.5 times the maximum stone diameter or 6 inches, whichever is greater (apron thickness = $1.5d_{max}$). Apron thickness may be reduced to $1.5d_{50}$ when an appropriate filter fabric is used under the apron.

c. The apron width at the discharge outlet shall be at least equal to the pipe diameter or culvert width (d_w). Riprap shall extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and shall taper to the flat surface at the end of the apron.

DRAFT

d. If there is a well-defined channel, the apron length shall be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel shall not be steeper than 2:1.

e. If the ground slope downstream of the apron is steep, channel erosion may occur. The apron shall be extended as necessary until the slope is gentle enough to prevent further erosion.

f. Ensure that the apron has zero grade. There shall be no overfall at the end of the apron; that is, the elevation of the top of the riprap at the downstream end shall be the same as the elevation of the bottom of the receiving channel or the adjacent ground if there is no channel.

g. The apron should be straight throughout its entire length, but if a curve is necessary to align the apron with the receiving stream, locate the curve in the upstream section of riprap.

h. Immediately after construction, stabilize all disturbed areas with vegetation.

i. The potential for vandalism shall be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased, or the rocks held in place using concrete or grout.

Installation

a. Ensure that the subgrade for the fabric and riprap follows the required lines and grades shown in the plan. Compact any fill required in the subgrade to the density of the surrounding undisturbed material. Low areas in the subgrade on undisturbed soil may also be filled by increasing the riprap thickness.

b. The riprap and fabric must conform to the specified grading limits shown on the plans.

c. Filter cloth must be properly protected from punching or tearing during installation. Repair any damage by removing the riprap and placing another piece of filter cloth over the damaged area. All connecting joints shall overlap a minimum of 1 foot. If the damage is extensive, replace the entire filter cloth.

d. Riprap may be placed by equipment but take care to avoid damaging the fabric.

Inspection & Maintenance Guidelines

a. Inspect riprap outlet structures after heavy rains to see if any erosion around or below the riprap has taken place or if stones have been dislodged. Immediately make all needed repairs to prevent further damage.

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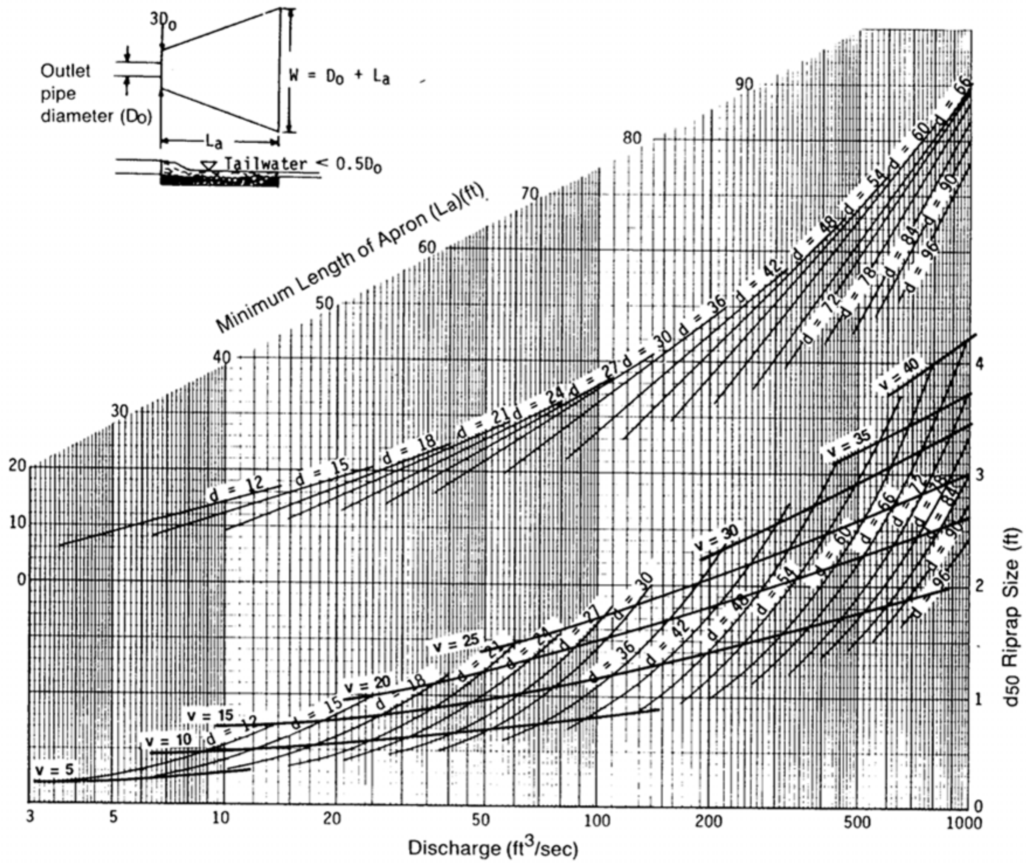
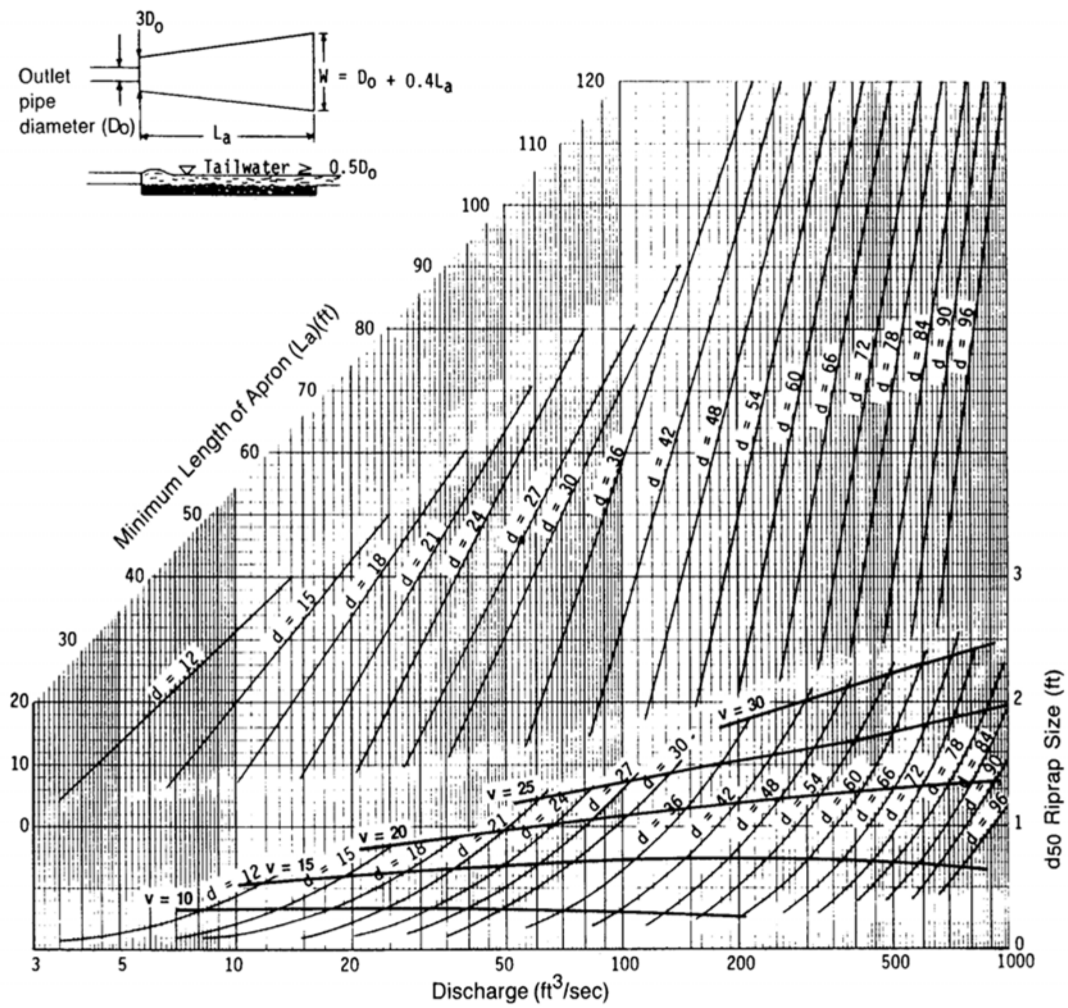
Figure 10-9. Design of Riprap Apron Under Minimum Tailwater Conditions



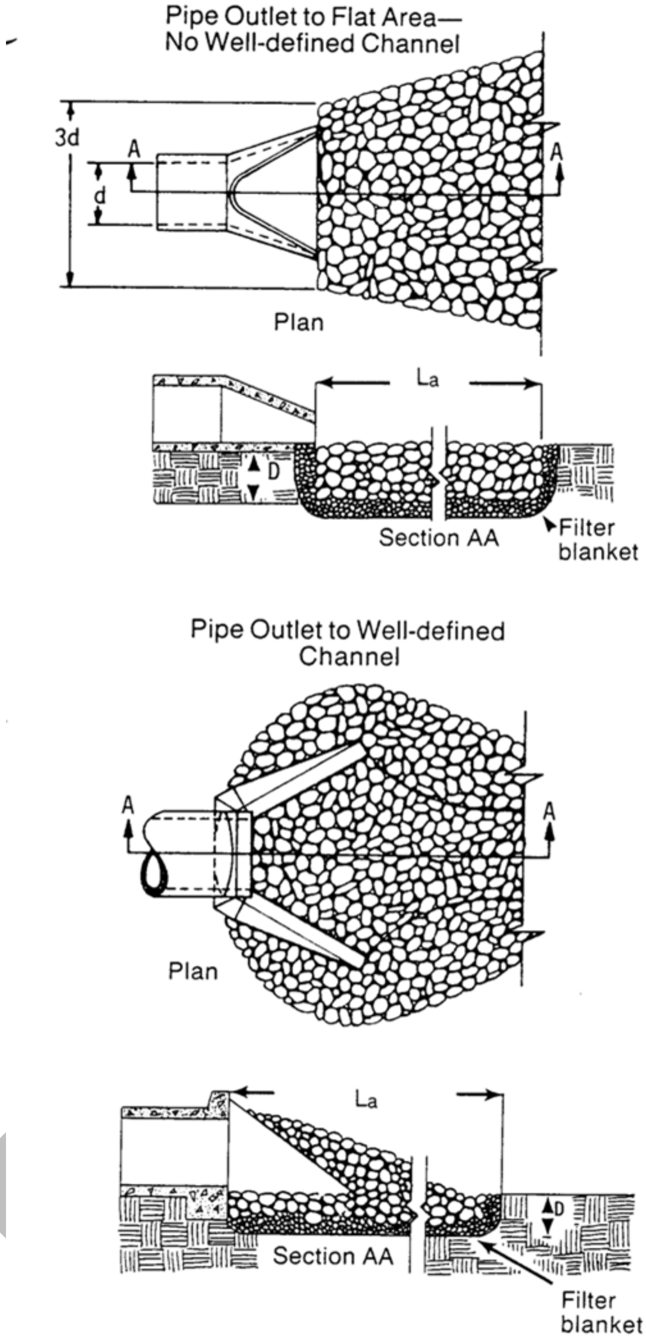
Curves may not be extrapolated.

Figure 10-10. Design of Riprap Apron Under Maximum Tailwater Conditions



Curves may not be extrapolated.

Figure 10-11. Riprap Apron



Notes

1. L_a is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter but not less than 6".
3. In a well-defined channel extend the apron up the channel banks to an elevation of 6" above the maximum tailwater depth or to the top of the bank, whichever is less.
4. A filter blanket or filter fabric should be installed between the riprap and soil foundation.

10.9.2 Riprap Stilling Basin

Description

A riprap stilling basin is a preshaped scour hole lined with riprap that functions as an energy dissipato***r*** by forming a hydraulic jump. General details of the basin are shown in Figure 10-12. Principle features of the basin are:

a. The basin is preshaped and lined with riprap of median size (d_{50}).

b. The floor of the riprap basin is constructed at a depth of h_S below the culvert invert. The dimension h_S is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge.

c. The ratio of h_S to d_{50} of the material should be between 2 and 4 ($2 < h_S/d_{50} < 4$).

d. The length of the energy dissipating pool is $10h_S$ or $3W_o$, whichever is larger. The overall length of the basin is $15h_S$ or $4W_o$, whichever is larger.

e. Layout details are shown on Figure 10-12.

High tailwater conditions ($T_w/d_o > 0.75$) have the following features.

a. The high velocity water emerging from the culvert retains its jet like character as it passes through the basin.

b. The scour hole is not as deep as with low tailwater and is generally longer.

c. Riprap may be required for the channel downstream of the rock-lined basin.

The following variables are used in the design of riprap basins.

A = flow area
d_o = y_o = normal flow depth
d_E = y_e = equivalent depth at the brink = $(A/2)^{0.5}$
d_{50} = median size of rock by weight; rounded or angular rock
Fr = Froude Number = $V/(gd_E)^{0.5}$
g = acceleration of gravity = 32.2 ft/s^2
h_S = approximate depth of scour
L_s = length of dissipating pool
L_B = length of basin
Q = design discharge
T_w = tailwater depth
V_o = velocity at the outlet
W_o = width of pipe or box culvert at the outlet

DRAFT

Design Procedure

The following procedure shall be used for the design of riprap basins.

Step 1: Determine Input Flow

a. d_o or d_E, V_o, and Fr at the culvert outlet.

Step 2: Check T_w

a. Determine if $T_w/d_o < 0.75$.

Step 3: Determine d_{50}

a. Use Figure 10-13.
b. Select d_{50}/d_E. Satisfactory results will be obtained if $0.25 < d_{50}/d_E < 0.45$.
c. Obtain h_S/d_E using Froude number (Fr) and Figure 10-13.
d. Check if $2 < h_S/d_{50} < 4$ and repeat until a d_{50} is found within the range.

Step 4: Size Basin

a. As shown in Figure 10-12.
b. Determine length of the dissipating pool, L_S. $L_S = 10h_S$ or $3W_o$ minimum.
c. Determine length of basin, L_B. $L_B = 15h_S$ or $4W_o$ minimum.
d. Thickness of riprap: Approach = $3d_{50}$ or $1.5d_{max}$, Remainder = $2d_{50}$ or $1.5d_{max}$.

Step 5: Determine the basin exit velocity (V_B)

a. Basin exit depth, d_B = critical depth at basin exit.
b. Basin exit velocity, $V_B = Q/(W_B d_B)$
c. Compare V_B with the average normal flow velocity in the natural channel, V_d.

Step 6: High Tailwater Design

a. Design a basin for low tailwater conditions, Steps 1-5.
b. Compute equivalent circular diameter D_E for brink area from:
$$A = (\pi/4)D_E^2 = d_o \times W_o$$
c. Estimate centerline velocity at a series of downstream cross sections using Figure 10-14.
d. Size riprap using the FHWA HEC 23.

Step 7: Design Filter

a. Unless the streambed material is sufficiently well graded.
b. Follow instructions in HEC 23.

DRAFT

Material, construction techniques, and design details for riprap shall be in accordance with specifications in the Federal Highway Administration's publication HEC-23. Previously, revetment design was found in HEC-11, which has been archived by FHWA. Also see Section 10.4.4 of this document for a summary of requirements for riprap design.

Design Guidelines

Riprap basin design shall include consideration of the following:

a. The dimensions of a scour hole in a basin constructed with angular rock can be approximately the same as the dimensions of a scour hole in a basin constructed of rounded material when rock size and other variables are similar.

b. When the ratio of tailwater depth to brink depth, T_w/d_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_S/d_{50}, is greater than 2.0, the scour hole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scour hole, and flow is generally well dispersed as it leaves the basin.

c. The mound of material formed on the bed downstream of the scour hole contributes to the dissipation of energy and reduces the size of the scour hole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scour hole will enlarge.

d. For high tailwater basins (T_w/d_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scour hole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

e. It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2d_{50}$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

f. See Standards in FHWS HEC 23 for details on riprap materials and use of filter fabric. Also see Section 10.4.4 of this document for a summary of requirements for riprap design.

g. Stability of the surface at the outlet of a basin shall be considered using the methods for open channel flow as outlined in Section 10.5.

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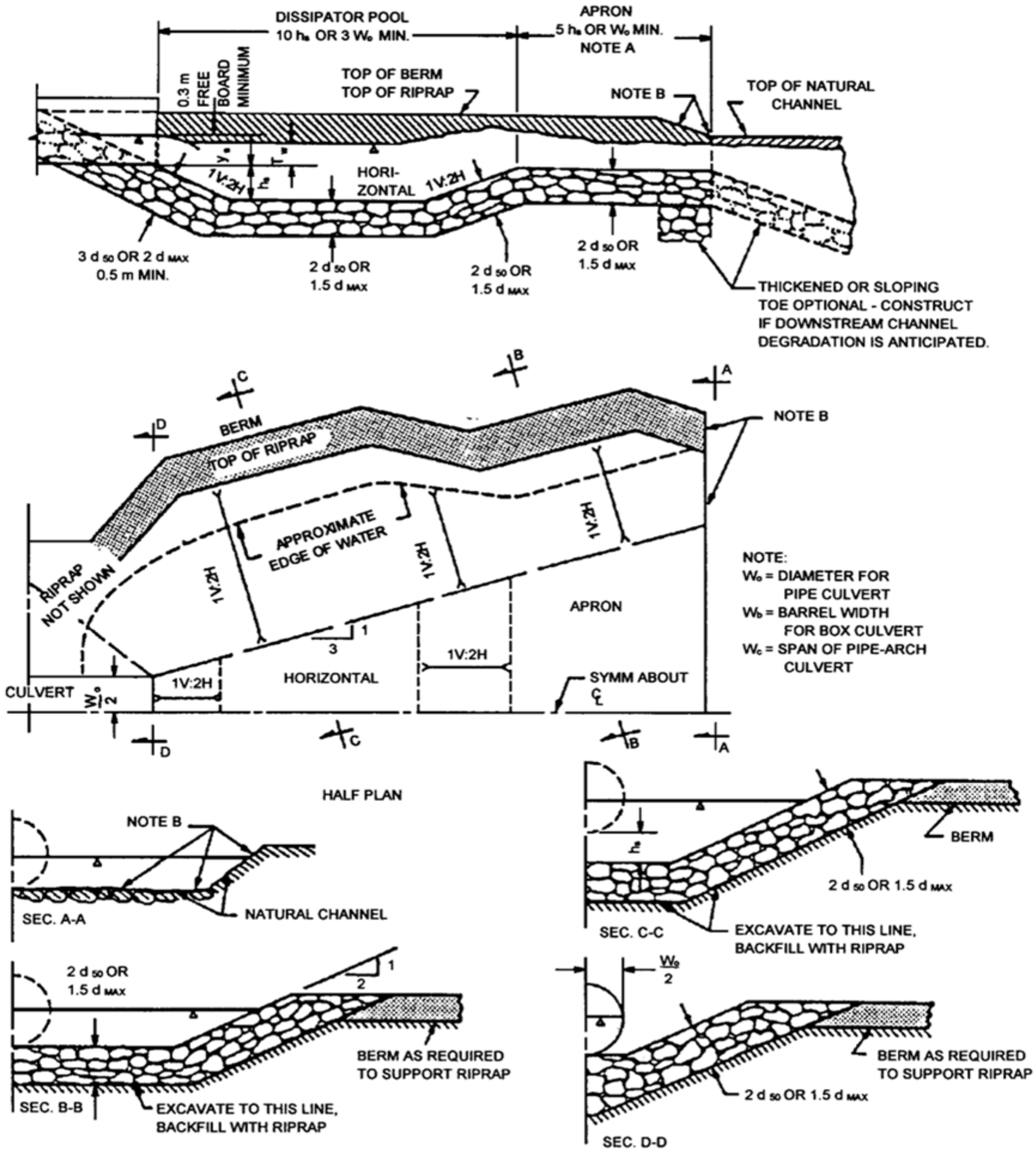
Figure 10-12. Details of a Riprap Stilling Basin



NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

Figure 10-13. Riprap Basin Depth of Scour

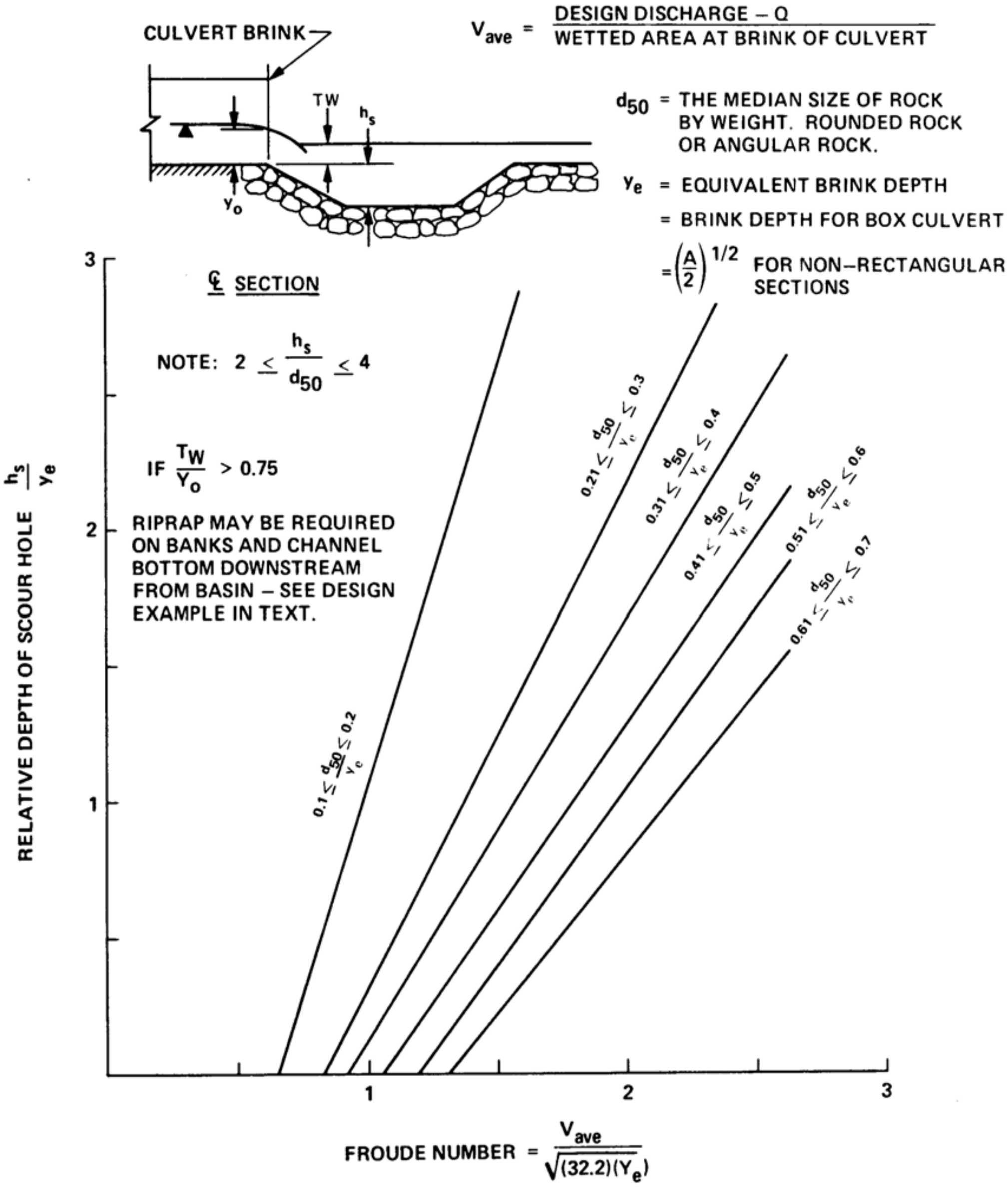
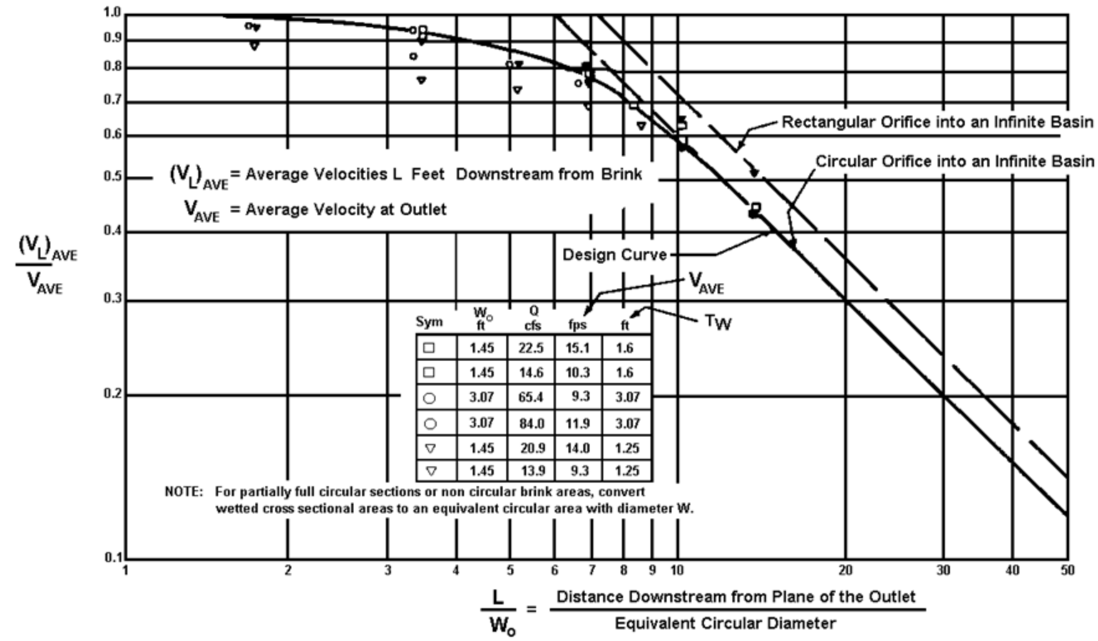


Figure 10-14. Estimating Centerline Velocity



Sym	W_o ft	Q cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
○	3.07	65.4	9.3	3.07
○	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

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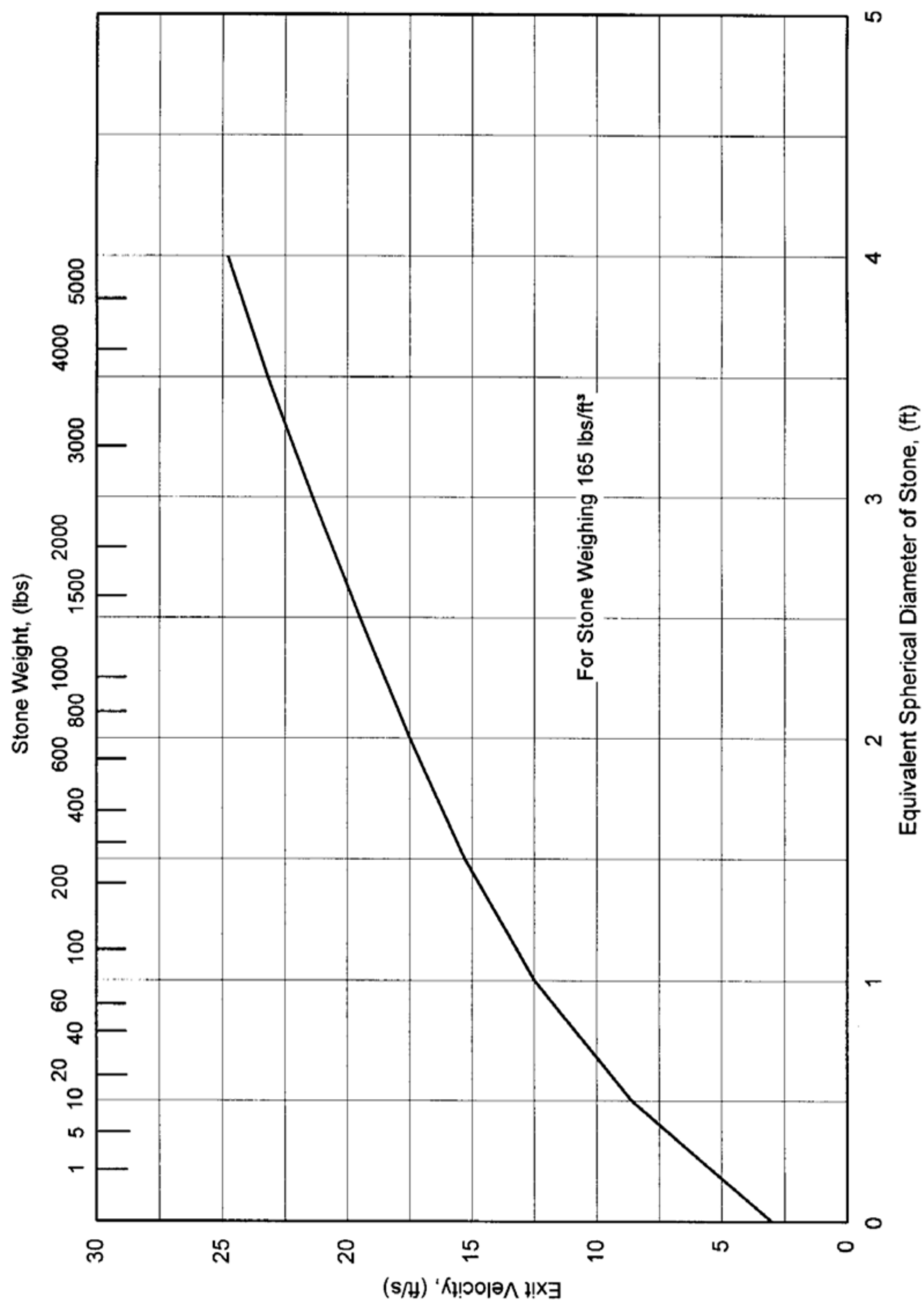
Figure 10-15. Riprap Size for Use Downstream of Energy Dissipator



19

10.9.3 Impact Basin - USBR Type VI

<u>Description</u>

The Impact Basin - USBR Type VI (also known as the baffled outlet) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 10-16. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

<u>Design Procedure</u>

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 10-20 shall be calculated as follows.

Step 1: Calculate equivalent depth, d_E

a. Rectangular section, $d_E = d_o = y_o$
b. Other sections, $d_E = (A/2)^{0.5}$ **Equation 10-2**

Step 2: Determine Input Flow

a. Froude number, $Fr = V_o(gd_E)^{0.5}$ **Equation 10-3**
b. *Specific energy,* $H_o = d_E + (V^2/2g)$ **Equation 10-4**

Step 3: Determine Basin Width

a. Use Figure 10-17
b. Enter with Fr and read H_o/W
c. $W = H_o/(H_o/W)$ **Equation 10-5**

Step 4: Size Basin

a. Use Table 10-4 and W
b. Obtain the remaining dimensions

Step 5: Energy Loss

a. Use Figure 10-18
b. Enter with Fr and read H_L/H_o
c. $H_L = (H_L/H_o)H_o$ **Equation 10-6**

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Step 6: Exit Velocity (V_B)

a. Exit energy (H_E)= $H_o - H_L$ **Equation 10-7**
b. $H_E = d_B + (V_B^2/2g)$ **Equation 10-8**
c. $V_B = (Q/W)/d_B$ **Equation 10-9**

Design Guidelines

a. Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies.

b. Notches are provided to aid in cleaning the basin. The notches provide concentrated jets of water for cleaning. The basin is designed to carry the full discharge over the top of the baffle if the space beneath the baffle becomes completely clogged.

c. The equivalent depth must be calculated for a pipe or irregular shaped conduit. The cross-section flow area in the pipe is converted into an equivalent rectangular cross section in which the width is twice the depth of flow.

d. Discharges up to 400 cfs per barrel and velocities as high as 50 ft/s can be used without subjecting the structure to cavitation damage.

e. A moderate depth of tailwater will improve performance. For best performance, set the basin so that maximum tailwater does not exceed $h_3 + (h_2/2)$.

f. If culvert slope is greater than 15°, a horizontal section of at least four culvert widths shall be provided upstream.

g. An end sill with a low-flow drainage slot, 45° wingwalls, and a cutoff wall shall be provided at the end of the basin.

h. Riprap shall be placed downstream of the basin for a length of at least four conduit widths.

i. If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent of diameter approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

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Figure 10-16. Schematic of Impact Basin or Baffled Outlet

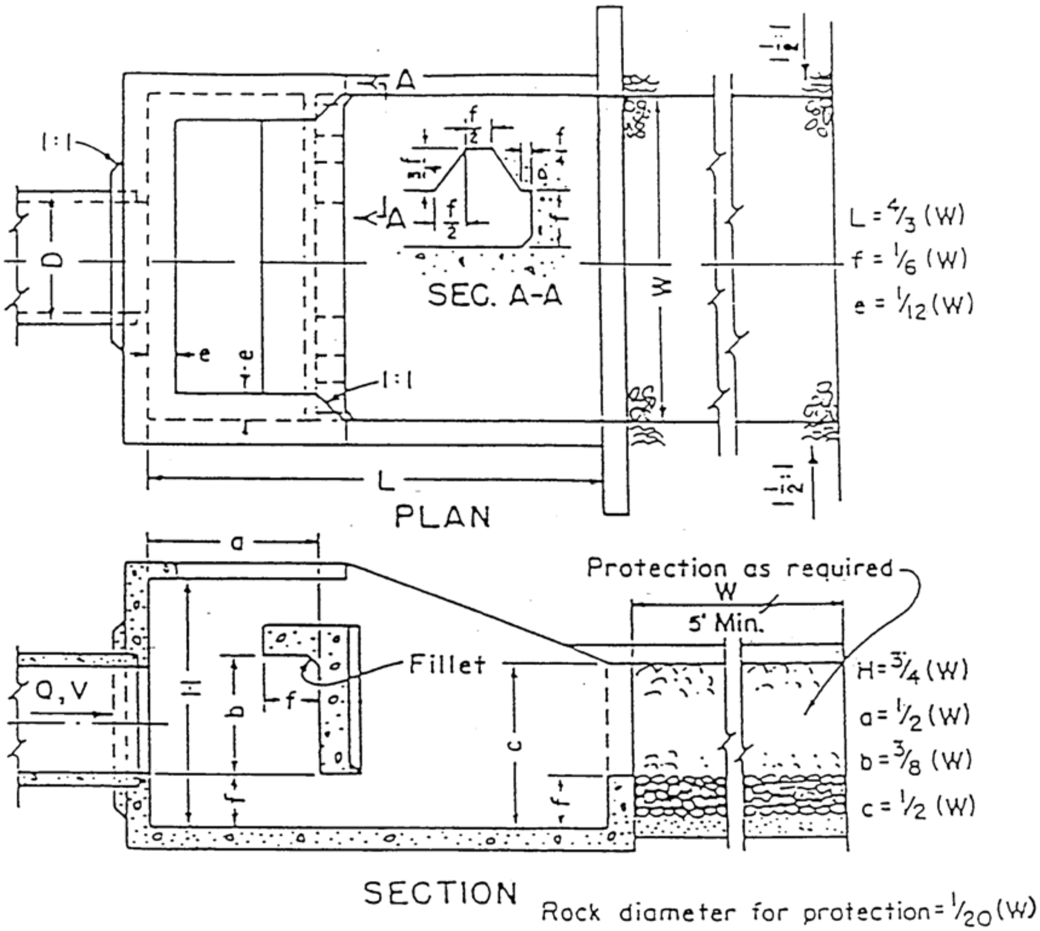

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Table 10-4. Dimensions of USBR Type VI Basin (ft)

W	h_1	h_2	h_3	h_4	L	L_1	L_2
4	3.08	1.50	0.67	1.67	5.42	2.33	3.08
5	3.83	1.92	0.83	2.08	6.67	2.92	3.83
6	4.58	2.25	1.00	2.50	8.00	3.42	4.58
7	5.42	2.58	1.17	2.92	9.42	4.00	5.42
8	6.17	3.00	1.33	3.33	10.67	4.58	6.17
9	6.92	3.42	1.50	3.75	12.00	5.17	6.92
10	7.67	3.75	1.67	4.17	13.42	5.75	7.67
11	8.42	4.17	1.83	4.58	14.58	6.33	8.42
12	9.17	4.5	2.00	5.00	16.00	6.83	9.17
13	10.17	4.92	2.17	5.42	17.33	7.42	10.00
14	10.75	5.25	2.33	5.83	18.67	8.00	10.75
15	11.5	5.58	2.50	6.25	20.00	8.50	11.50
16	12.25	6.00	2.67	6.67	21.33	9.08	12.25
17	13.00	6.33	2.83	7.08	21.50	9.67	13.00
18	13.75	6.67	3.00	7.50	23.92	10.25	13.75
19	14.58	7.08	3.17	7.92	25.33	10.83	14.58
20	15.33	7.5	3.33	8.33	26.58	11.42	15.33

W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
4	0.33	1.08	0.50	0.50	0.50	0.50	0.25
5	0.42	1.42	0.50	0.50	0.50	0.50	0.25
6	0.50	1.67	0.50	0.50	0.50	0.50	0.25
7	0.50	1.92	0.50	0.50	0.50	0.50	0.25
8	0.58	2.17	0.50	0.58	0.58	0.50	0.25
9	0.67	2.50	0.58	0.58	0.67	0.58	0.25
10	0.75	2.75	0.67	0.67	0.75	0.67	0.25
11	0.83	3.00	0.67	0.75	0.75	0.67	0.33
12	0.92	3.00	0.67	0.83	0.83	0.75	0.33
13	1.00	3.00	0.67	0.92	0.83	0.83	0.33
14	1.08	3.00	0.67	1.00	0.92	0.92	0.42
15	1.17	3.00	0.67	1.00	1.00	1.00	0.42
16	1.25	3.00	0.75	1.00	1.00	1.00	0.50
17	1.33	3.00	0.75	1.08	1.00	1.00	0.50
18	1.33	3.00	0.75	1.08	1.08	1.08	0.58
19	1.42	3.00	0.83	1.17	1.08	1.08	0.58
20	1.50	3.00	0.83	1.17	1.17	1.17	0.67

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Figure 10-17. Design Curve for USBR Type VI Dissipator

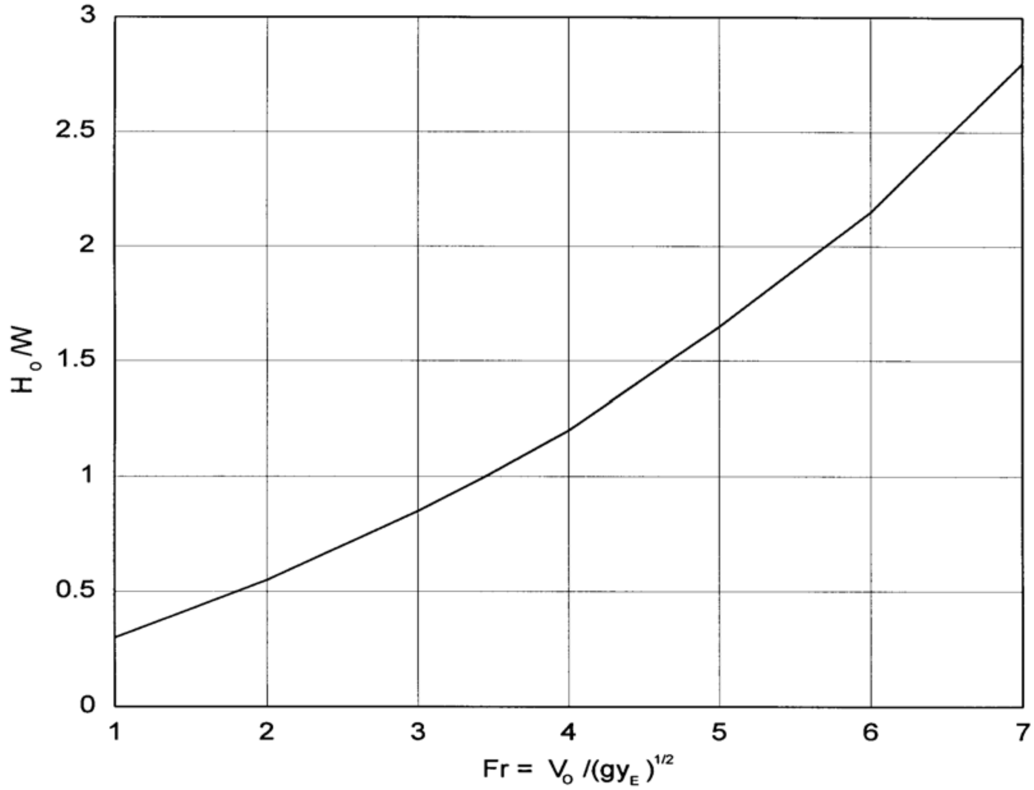

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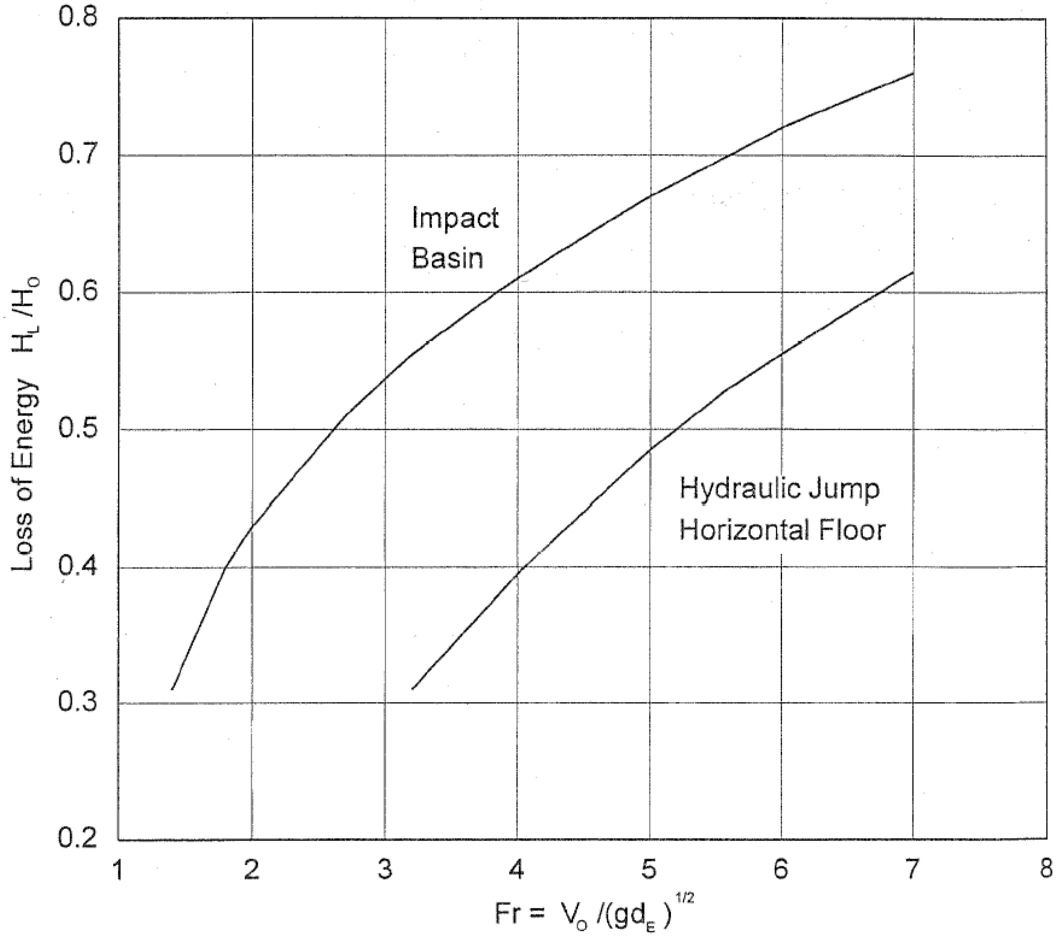
Figure 10-18. Energy Loss for USBR Type VI Dissipator



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10.9.4 SAF Stilling Basin

<u>Description</u>

The St. Anthony Falls (SAF) stilling basin uses a forced hydraulic jump to dissipate energy and is based on model studies conducted by the U.S. Soil Conservation Service (SCS) at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota. It uses chute blocks, baffle blocks, and an end sill to force the hydraulic jump and reduce jump length by about 80%, and it is recommended where Fr = 1.7 to 17. Figures 10-19 and 10-20 show the dimensions that will need to be determined when designing the basin. Figure 10-23 illustrates examples of several other stilling basin designs.

The following equations will be used during the design procedure:

Basin Width, W_B

- For a box culvert, W_B = B = culvert width (ft)
- For pipe, W_B = D = culvert diameter (ft), or

$$W_B = \left(\frac{0.03Q}{D^{1.5}}\right)$$ **Equation 10-10**

where Q = discharge (cfs)
Use whichever is larger.

Flare, Z:1

- Flare is optional, if used it shall be flatter than 2:1

Basin Length, L_B

$$d_j = \frac{d_1}{2}\left[(1 + 8Fr_1^2)^{0.5} - 1\right]$$ **Equation 10-11**

where d_1 = initial depth of water (ft);
d_j = sequent depth of jump (ft)
Fr_1 = Froude number entering basin, ≠ Fr

$$L_B = \frac{4.5d_j}{Fr_1^{0.76}}$$ **Equation 10-12**

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Basin Floor

- The basin floor should be depressed below the streambed enough to obtain the following depth (d_2) below the tailwater

For Fr_1 = 1.7 to 5.5

$$d_2 = d_j \left[1.1 - \left(\frac{Fr_1^2}{120}\right)\right]$$ **Equation 10-13**

For Fr_1 = 5.5 to 11

$$d_2 = 0.85 d_j$$ **Equation 10-14**

For Fr_1 = 11 to 17

$$d_2 = d_j \left[1.1 - \left(\frac{Fr_1^2}{800}\right)\right]$$ **Equation 10-15**

Chute Blocks

- Height, $h_1 = d_1$ **Equation 10-16**
- Width, W_1 = Spacing = $0.75d_1$ **Equation 10-17**
- Number of blocks = $N_c = W_B/2W_1$, rounded to a whole number **Equation 10-18**
- Adjusted $W_1 = W_2 = W_B/2N_c$ **Equation 10-19**
- N_c includes the ½ block at each wall

Baffle Blocks

- Height, $h_3 = d_1$ **Equation 10-20**
- Width, W_3 = Spacing = $0.75d_1$ **Equation 10-21**
- Basin width at baffle blocks, $W_{B2} = W_B + 2L_B/3Z$ **Equation 10-22**
- Number of blocks = $N_B = W_{B2}/2W_3$, rounded to a whole number **Equation 10-23**
- Adjusted $W_3 = W_4 = W_{B2}/2N_B$ **Equation 10-24**
- Check total block width to insure that 40% to 55% of W_{B2} is occupied by block.
- Staggered with chute blocks
- Space at wall ≥ $0.38d_1$
- Distance from chute blocks (L_{1-3}) = $L_B/3$ **Equation 10-25**

End Sill Height

- $h_4 = 0.07d_j$ **Equation 10-26**

Sidewall Height

- $h_5 = d_2 + 0.33d_j$ **Equation 10-27**

Wingwall Flare

- = 45°

Energy Equation

- $Q = d_1 W_B \sqrt{2g(z_o - z_1 + d_o - d_1) + V_o^2}$ **Equation 10-28**
- $V_1 = \frac{Q}{d_1 W_B}$ **Equation 10-29**

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<u>Design Procedure</u>

The design of an SAF basin consists of the following steps.

Step 1: Compute the culvert brink depth ($y_0 = d_0$) using Figures 10-21 and 10-22.

Step 2: Compute the tailwater depth ($T_w = y_n$) in the downstream channel assuming normal flow (using Manning's equation – Equation 3-1) or perform backwater analysis.

Step 3: Determine the basin elevation by selecting z_1 and then compute the following:

a. Determine basin width (W_B) using Equation 10-10 and select basin slopes S_X and S_T (see Figure 10-19)
b. Check W_B using

$$W_B < W_o + \left[\frac{2L_T\sqrt{S_T^2+1}}{3Fr_o}\right]$$

$$Where\ L_T = \frac{Z_o - Z_1}{S_T}$$

c. Compute y_1 (or d_1) using Equations 10-28 and 10-29
d. Compute Fr1 using $Fr_1 = \frac{V_1}{\sqrt{gd_1}}$ **Equation 10-30**

Step 4: Calculate Basin Dimensions

a. d_j (Equations 10-11 and 10-30)
b. L_B (Equation 10-12)
c. d_2 (Equation 10-13, 10-14, or 10-15)
d. $L_S = (d_2 - T_w)/S_S$ (see Figure 10-19)
e. $L_T = (z_0 - z_1)/S_T$ (see Figure 10-19)
f. $L = L_T + L_B + L_S$ (see Figure 10-19)
g.

$$Z_3 = \frac{Z_o - \left(L_T + L_B - \frac{Z_2}{S_S}\right)S_o}{\frac{S_o}{S_S}+1}$$

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Step 5 Review Results

a. If $d_2 \neq (T_w + z_3 - z_2)$ return to Step 2
b. If approximately equal, continue

Step 6: Size Elements

a. Chute blocks: h_1, W_1, W_2, and N_c, using Equations 10-16 through 10-19
b. Baffle blocks: h_3, W_3, W_4, N_B, $L_{1\text{-}3}$, using Equations 10-20 through 10-25
c. End sill: h_4, using Equation 10-26
d. Side wall height: h_5, using Equation 10-27

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Figure 10-19. Hydraulic Jump Basin

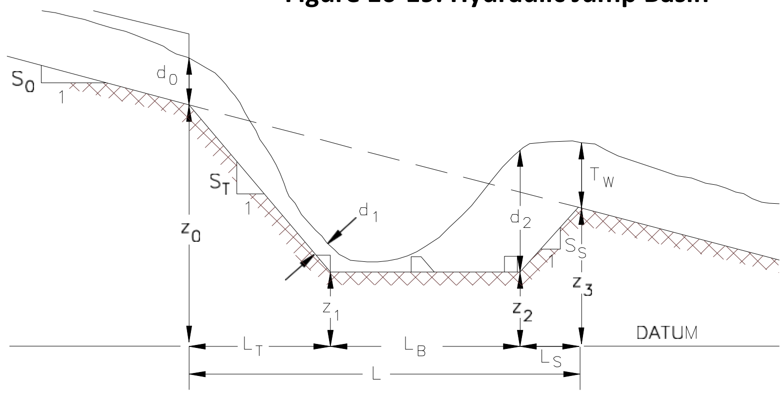


Figure 10-20. St. Anthony Falls Basin

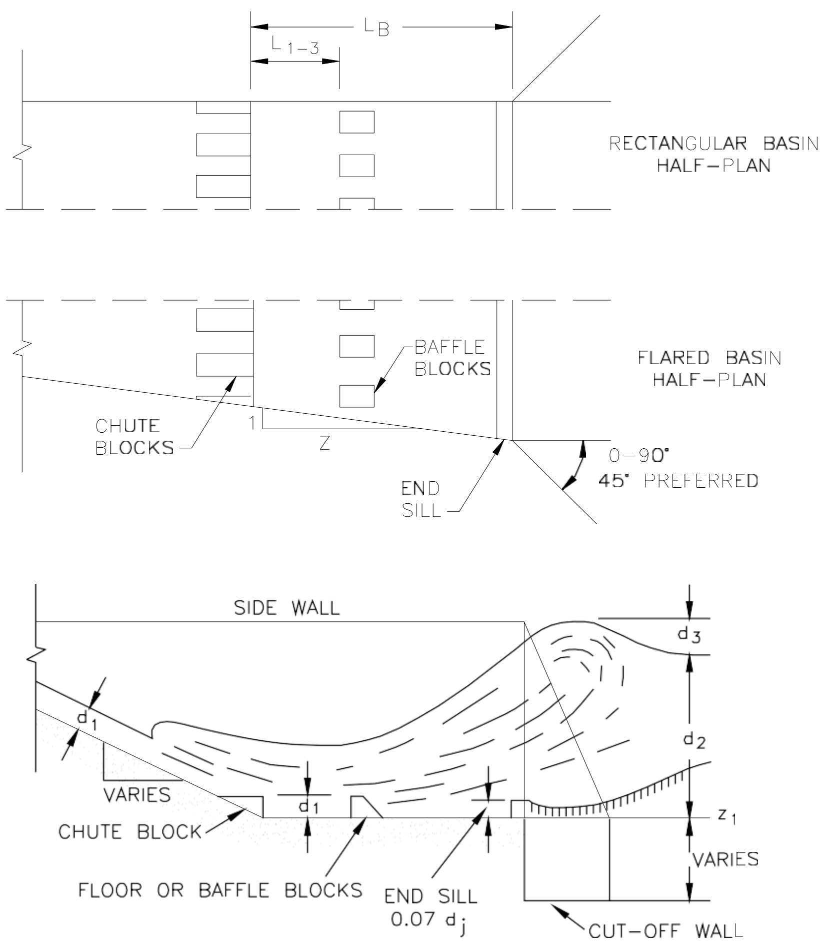


Figure 10-21. Dimensionless Rating Curves – Rectangular Culverts on Mild Slopes

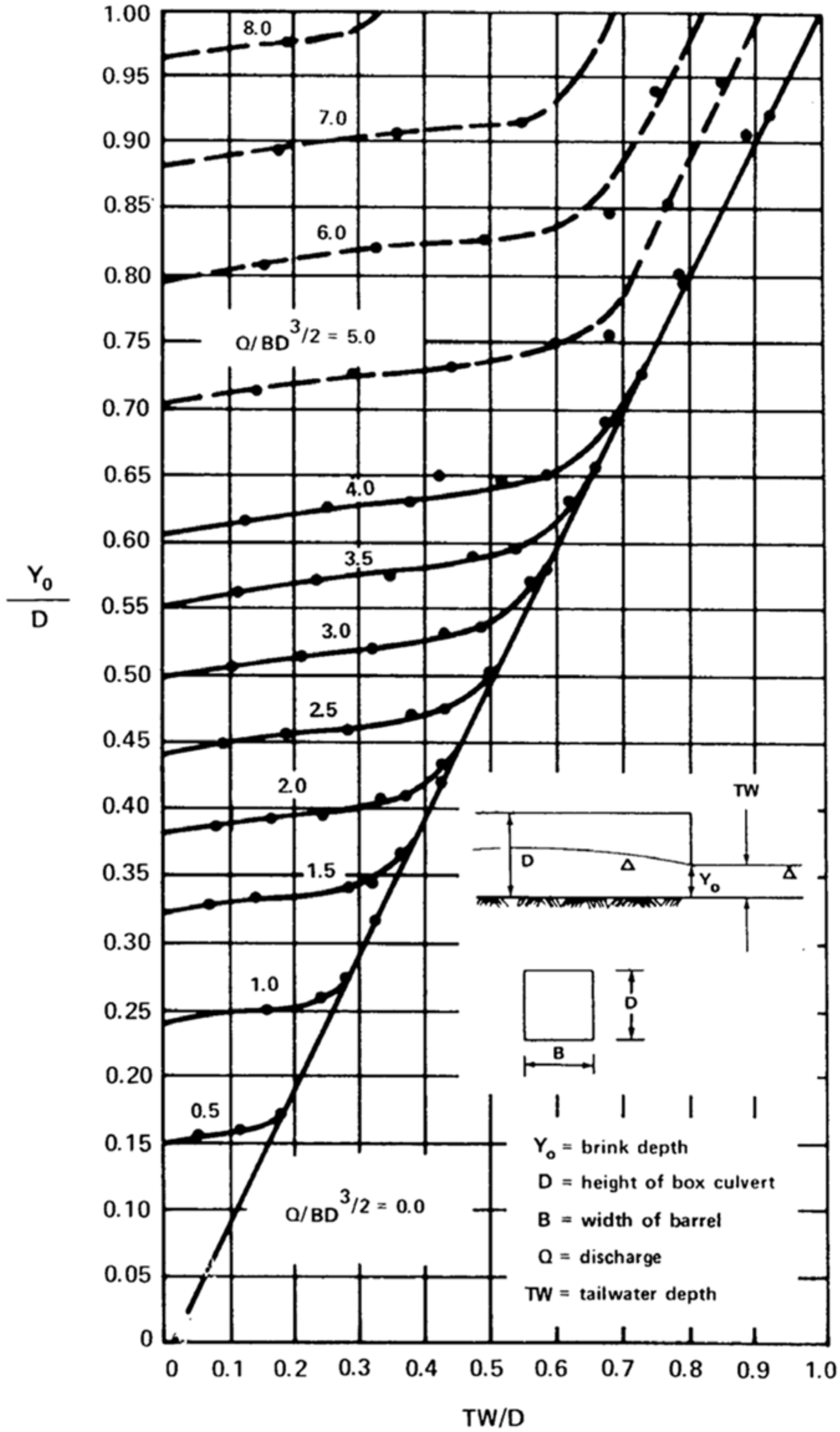


Figure 10-22. Dimensionless Rating Curves - Circular Culverts on Mild Slopes

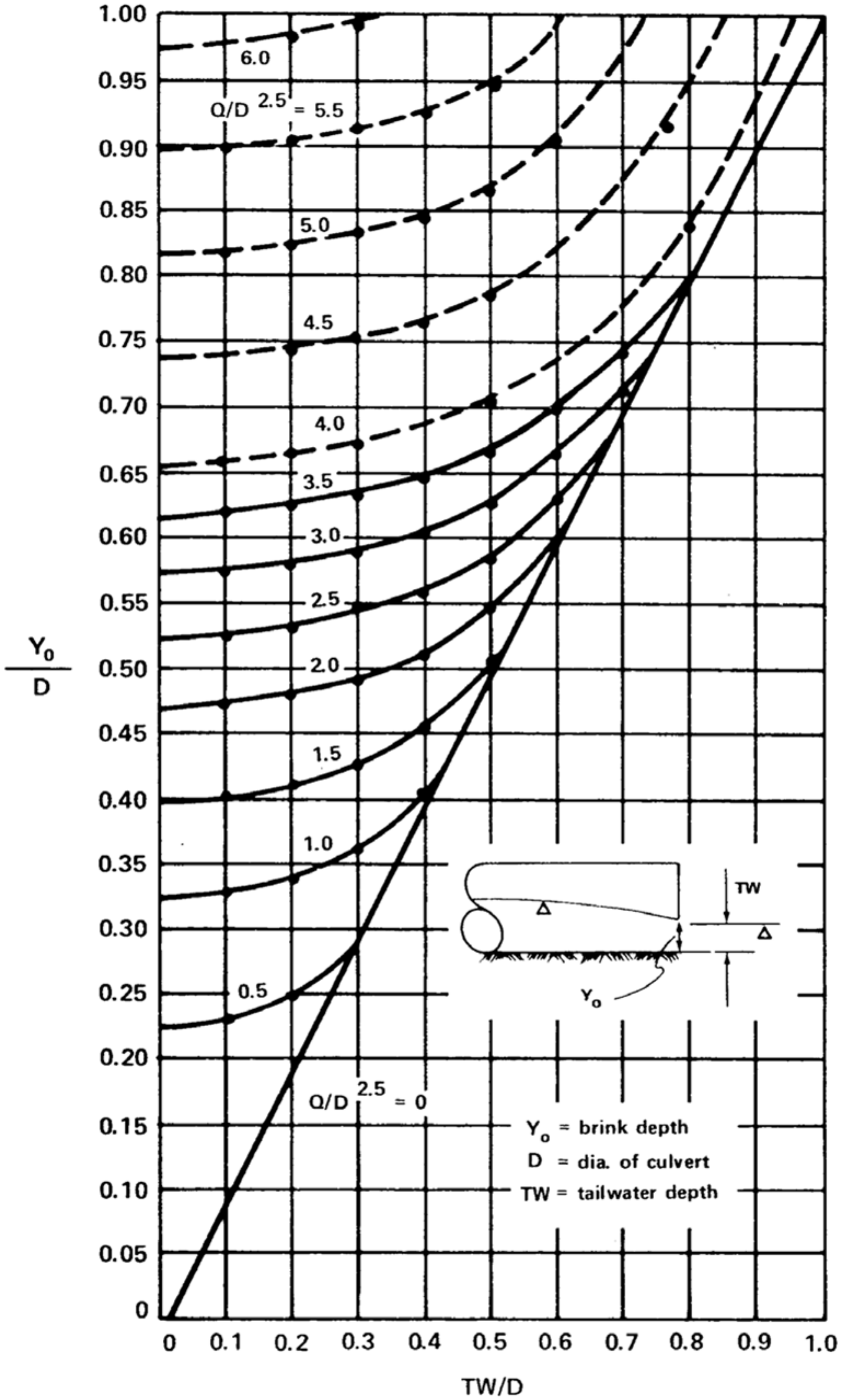
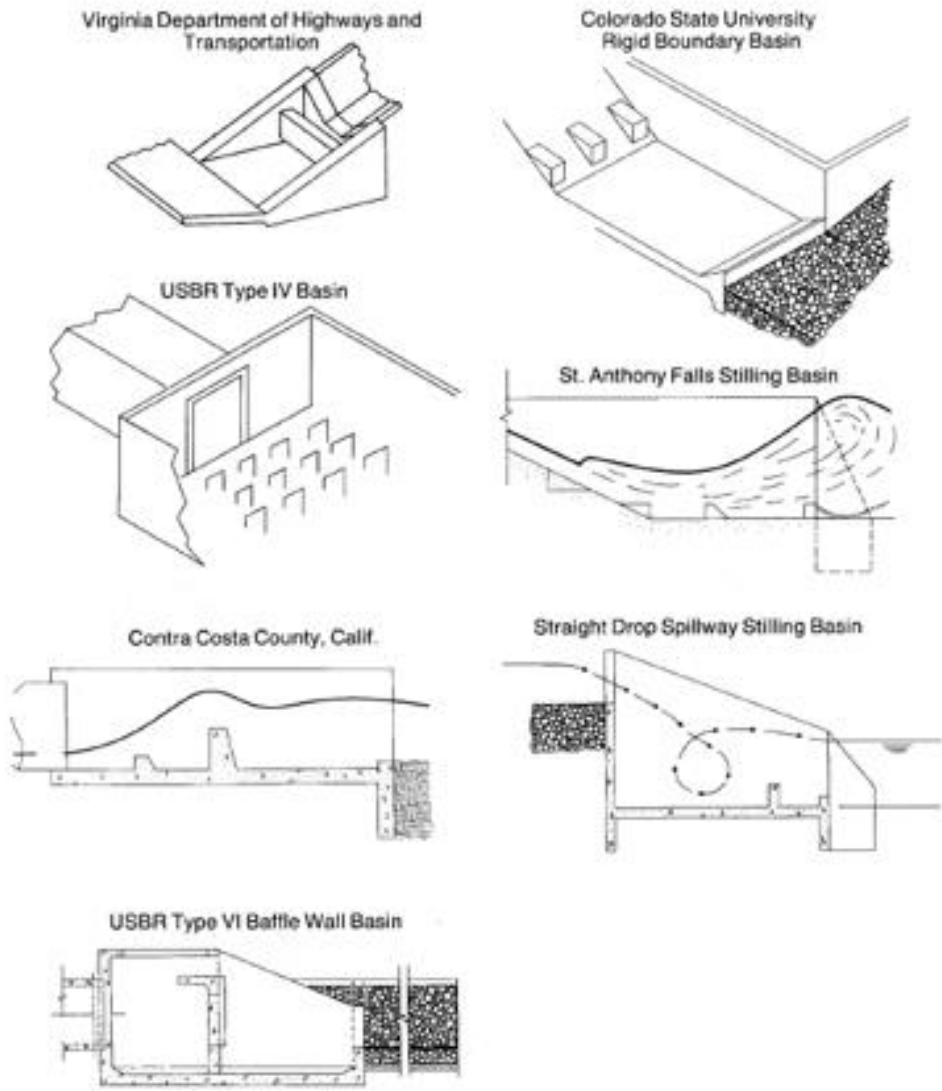


Figure 10-23. Examples of Stilling Basin Design Concepts



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Chapter 11
BEST MANAGEMENT PRACTICES (BMPs)

11.1 INTRODUCTION

The purpose of this chapter is to provide information and guidance regarding the selection, design, and use of Best Management Practices (BMPs) in the construction of storm water facilities. BMPs are measures that function to keep pollutants from entering the storm water or remove pollutants from storm water. The City of Corpus Christi has established procedures and criteria under their MS4 Permit and *Storm Water Quality Management: Guidance Document for Development Planning and Construction Activities*. However, it is recommended that the user verify with the City that any BMPs selected from the *Storm Water Quality Management* document are currently applicable.

In general, BMPs can be categorized as either structural or nonstructural. Structural BMPs are constructed facilities designed to reduce runoff and/or passively treat urban storm water runoff before it enters the receiving waters. Nonstructural BMPs consist of pollution prevention BMPs and source control BMPs. Both structural and nonstructural BMPs are used for erosion control during and after construction (see chapter 10).

The selection of the most appropriate BMPs for a given site or basin is largely dependent on whether development is in place or has yet to occur. In areas with existing development, nonstructural BMPs are encouraged because retrofitting structural controls in a developed area can be expensive. Structural controls are more appropriate for new development and significant redevelopment, where they can be integrated into the planning of the infrastructure.

Because non-point source pollution is varied in nature and impact, no individual BMP may fit all situations. A selection of BMPs should be tailored to fit the needs of particular sources and circumstances. An effective strategy for minimizing storm water pollution loads is to use multiple BMPs (structural, nonstructural, and source controls). Multiple BMPs and combining BMPs in series can provide complementary water quality enhancement that minimizes pollutant loads being transported to the receiving waters.

11.2 GENERAL WATER QUALITY MANAGEMENT APPROACH

Success in applying any management practice depends on selecting the appropriate option for the control objectives, specific conditions at the site, proper implementation, and maintenance. Up- to-date information regarding green infrastructure can be found at the EPA's Green Infrastructure resource center website (https://www.epa.gov/green-infrastructure/epa-green-infrastructure-resources). The following general planning and design guidelines for structural and nonstructural best management practices are recommended when developing a water quality control strategy:

a. Promote natural infiltration of urban runoff by minimizing onsite impervious areas and preserving natural, broad drainageways.

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b. Minimize directly connected impervious areas by providing grassed or gravel buffer zones between impervious surfaces. Divert runoff from impervious areas to pervious surfaces before the flows enter surface drainageways.

c. Locate structural BMPs in areas that avoid creating a nuisance and the need for increased maintenance.

d. Provide multiple accesses to facilities to improve maintenance capabilities.

e. Direct offsite storm water flow around the onsite facilities.

f. Revegetate and/or stabilize all areas disturbed by construction activities and all drainageways created as a part of the development.

g. Ensure the plantings and grass cover are firmly established before the Owner/Developer's obligation is released and maintenance efforts begin.

11.3 NONSTRUCTURAL BEST MANAGEMENT PRACTICES

Section 11.4 of this chapter presents the details of structural best management practices and their use within the municipal drainage system. The other main category of BMPs includes the many nonstructural or source control practices that can be used for pollution prevention and control of pollutants. In most cases it is much easier and less costly to prevent pollutants from entering the drainage system than trying to control pollutants with structural BMPs. Thus, the nonstructural BMPs shall be the first line of defense in protecting the receiving stream within the municipality. If used properly, the nonstructural BMPs can be very effective in controlling pollutants and greatly reduce the need for structural BMPs. In addition, nonstructural BMPs tend to be less costly, easier to design and implement, and easier to maintain than structural BMPs.

The *Storm Water Quality Management* (SWQM) document, published by the City of Corpus Christi, discusses long-term management controls. These are nonstructural controls that are primarily management-based practices which are designed to prevent or reduce the potential of storm water runoff contact with pollution-causing activities. These practices occasionally require constructed facilities or vegetated practices that are designed to reduce pollutant levels in storm water runoff. Where applicable, these practices are encouraged to be used by owners of individual residences, residential developments, commercial and institutional developments, and various industries. Applicable non-structural controls include:

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- Household Hazardous Material Storage & Disposal
- Litter Control
- Landscaping Practices
- Fertilizer & Pesticide Use
- Fueling Station Practices
- Vehicle Equipment Washing & Steam Cleaning Practices
- Street Sweeping
- Liquid Materials Loading & Unloading
- Liquids Storage in Aboveground Tanks Practices
- Container Storage of Liquids, Food Wastes, & Hazardous Wastes
- Spill Prevention Containment & Countermeasure Plan
- Outdoor Storage Practices
- Inlet Stenciling
- Rainwater Harvesting/Reuse (Rain Barrels and Cisterns)
- Installation of Rain Gardens & Bioretention Areas
- Protection of Riparian Buffers
- Disconnection of Impervious Surfaces Directing Rooftop Gutter Downspouts into Vegetated or Other Pervious Areas

11.4 STRUCTURAL BEST MANAGEMENT PRACTICES

11.4.1 Pollutant Removal Mechanisms

Although runoff may contain many individual pollutants, the pollutants can be grouped into two categories: particulate and soluble. Often, pollutants such as metals and oxygen demanding compounds become adsorbed or attached to particulate matter. Therefore, if the particulate matter is removed, so are the adsorbed constituents. There are four basic pollutant removal or immobilization mechanisms promoted by the BMPs described in this chapter. The following is an overview of each of them.

- **Sedimentation** - Particulate matter is settled out of urban runoff. Approximately 80 percent of metals in storm water are adsorbed to particles that are under 60 microns in diameter (i.e., fine silts and clays). Consequently, these particles can require long periods of time to settle out of suspension. With extended detention, however, the smaller particles can agglomerate into larger ones, thus removing a larger proportion of them through sedimentation.

- **Filtering** - Particulates can be removed from water by filtration. Filtration removes particles by attachment to small-diameter collectors such as sand.

- **Infiltration** - As surface runoff infiltrates or percolates into the ground, pollutant loads are removed or reduced in the runoff. Particulates are removed at the ground surface by filtration, and soluble contaminants can be adsorbed to the soil matrix as the runoff percolates into the ground. Soil characteristics such as permeability, cation exchange capacity, and depth to groundwater or bedrock limit the effectiveness of infiltration as a pollutant removal mechanism.

- **Biological Uptake** - Soluble constituents can be ingested or taken up from the water column and concentrated through bacterial action and phytoplankton growth. In addition, certain biological activities can reduce toxicity of some pollutants.

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11.4.2 Structural BMP Selection

Selecting the appropriate BMP for a site depends on several factors, including:

- The permeability and type of soil underlying the BMP;
- The size of the tributary basin and the generated runoff volume in relation to the size of the BMP;
- The slopes and geometry of the site;
- The amount of base flow;
- The proximity of bedrock to the surface;
- The proximity to the seasonal high groundwater table to the surface;
- Tributary basin land uses; and,
- The ability to handle high sediment loads.

11.4.3 Water Quality Control Volume

For many BMPs, combining the water quality facility with a flood control facility is practical and cost effective. Specifically, the water quality control volume (WQCV) that is recommended for control is the first half inch (0.5 inches) of runoff from the basin tributary to the BMP. For facilities that combine water quality control with flood control, the runoff from the design storms for the flood control criteria should be "stacked" on top of the water quality control volume. The water quality control volume shall be detained over at least a 24-hour period, and preferably for longer.

11.4.4 Structural BMP Descriptions

Design criteria and minimum standards for temporary BMPs can be found in chapter 4 of the SWQM document. The chapter also includes descriptions and discussions of individual BMPs for the following:

- Diverting flow;
- Managing overland flow;
- Trapping sediment in channelized flow;
- Establishing permanent drainageways;
- Protecting inlets;
- Trapping sediment during site dewatering;
- Preventing tracking;
- Other source controls on construction projects; and,
- Long-term management controls.

This section of the *DDM* gives information regarding the applicability, advantages, disadvantages, costs and maintenance considerations for permanent structural BMPs that may be used within the City of Corpus Christi. Other structural BMPs may also be applicable for use in the City, such as City low flow demonstration projects along the Oso Creek corridor. The Texas Natural Resources Conservation Commission has published the document *Complying with the Edwards Aquifer Rules: Technical Guidance*

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on Best Management Practices, which provides detailed information on various BMPs. The structural BMPs covered in this section include:

- Extended Dry Detention Basins
- Retention (Wet) Ponds
- Constructed Wetlands
- Grassed Swales
- Filter Strips & Flow Spreaders
- Sand Filters
- Infiltration Trenches
- Porous Pavement
- Oil/Grit Separators
- Grate Inlet Inserts
- Litter Traps

11.4.4.1 Extended Dry Detention Basins

Extended dry detention (EDD) basins are designed to completely empty at some time after storm water runoff ends. These are adaptations of the detention basins used for flood control. The primary difference is in outlet design; the extended basin uses a much smaller outlet that extends the retention time for more frequent events so that pollutant removal is facilitated. A 40-hour drain time for the WQCV is recommended to remove a significant portion of fine particulates and provide stream bank erosion control. The term "dry" implies that there is no significant permanent water storage.

Many designers encourage a two-stage design in which the upper stage is dry except for infrequent large storm events and the lower stage is regularly inundated, with a volume equal to the runoff from the mean storm (see Figure 11-1 for a representative schematic).

Typical applications

a. Significant areal requirement limits use; not typically a site-based BMP.

b. Retrofitting to established developments may be very difficult due to areal requirements.

c. Extended dry detention basins can reduce peak storm water runoff rates while trapping sediment loads, particularly when used downstream from construction sites. Sediment from such high loads will need to be removed, however.

d. Extended dry detention basins can be used to improve runoff water quality from roads, parking lots, residential, commercial and industrial areas. Typically, they are used in conjunction with other onsite BMPs.

DRAFT

Design Considerations

a. See Chapter 7 for design criteria for detention basins.

Advantages

a. Moderate to high removal of particulates and suspended heavy metals.

b. Infiltration and resultant recharge to ground water is minimal compared to infiltration type BMPs; therefore, the risk of direct introduction of contaminants to ground water is also minimal.

Disadvantages

a. Possible habitat destruction.

b. High ground water levels may inundate the basin and outlet (use retention ponds if this is the case); ground water mounding may occur with slow-draining or silt-clogged soils.

c. Thermal modification to downstream waters is possible but should be minimal.

d. Will likely have negative aesthetics unless a lower-stage basin is used.

e. Can become a trash dump if not maintained.

f. Debris and sediment accumulation and removal, as well as overall design integration with the site must be factored into the design.

g. Potential breeding grounds for mosquitoes and other insects unless a balanced habitat is established.

Maintenance/Monitoring/Enforcement Considerations

a. Exfiltration will tend to decrease over time as the bottom becomes clogged with sediment; this may be a positive factor in preventing ground water contamination.

b. Cleanup of debris and trash, pest and overgrowth control, erosion repairs, inspect for structural damage to outlets, clogging of outlet.

c. A five year sediment cleanout cycle is recommended.

d. Regular maintenance and sediment cleanout are not technically difficult; long-term management should not be problematic.

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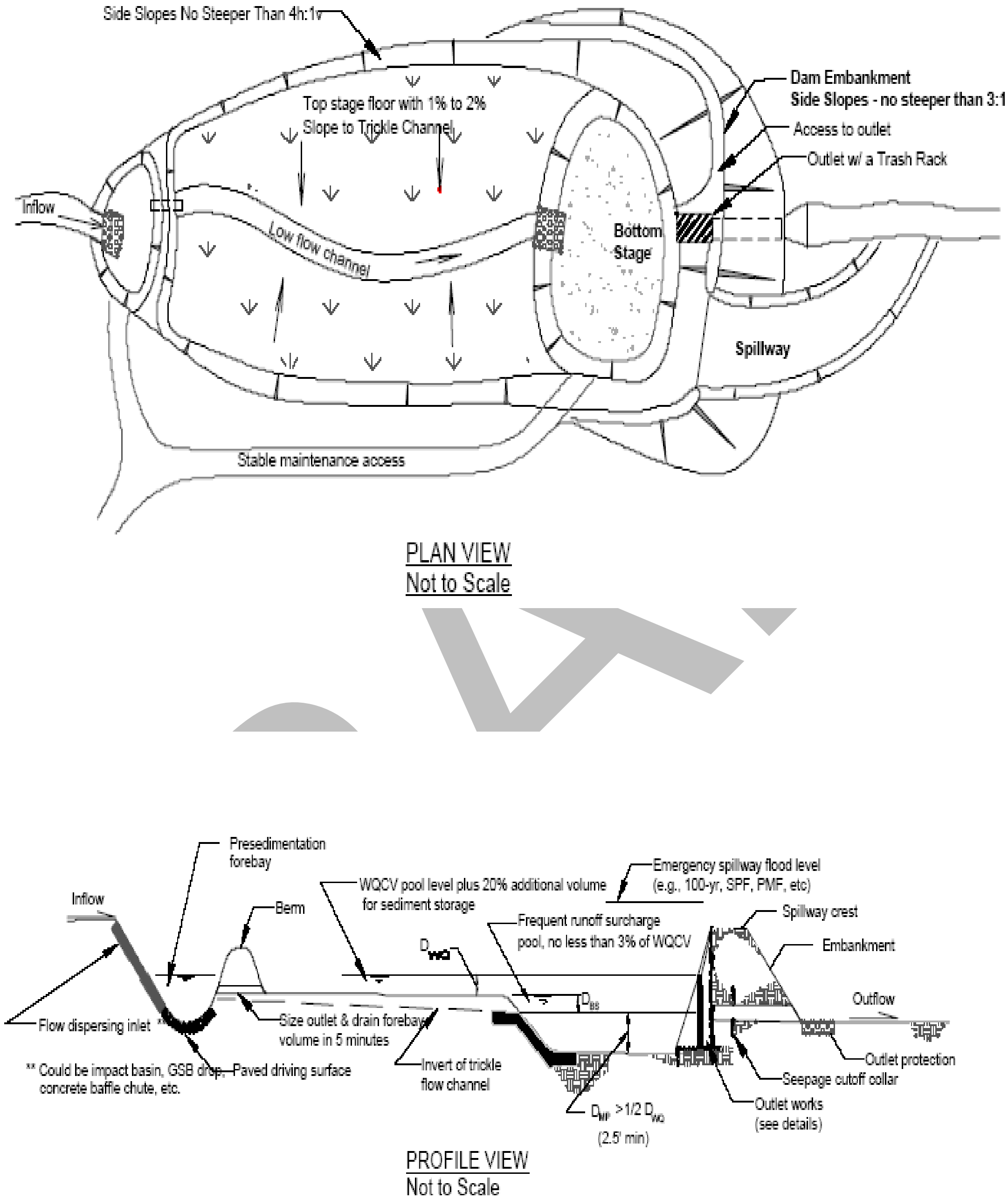
Figure 11-1. Extended Dry Detention Basin



11.4.4.2 Retention (Wet) Ponds

A retention pond is designed to not completely drain as in the dry basin design. A permanent pool of water is replaced in part by storm water during an event. For water quality purposes, the design is such that the WQCV is released over 12 to 24 hours, but the hydraulic residence time (HRT) for the permanent pool volume is two weeks or longer (see Figures 11-2 and 11-3 for representative schematics). Reduction of volume in the permanent pool is through evapotranspiration and infiltration only. A dry weather base flow may be required to maintain the permanent pool.

<u>Typical Applications</u>

a. Typically, not a site-based BMP, but retention ponds are effective in most settings where adequate open area exists. Due to the area required, it is difficult to retrofit to a completely developed watershed.

b. Since evaporation can quickly dry up base flows, retention ponds work best in areas with low evapotranspiration rates and/or non-arid climates.

c. Wet retention ponds can reduce peak storm water runoff rates while trapping sediment loads, as well as provide some biological uptake of nutrients. They can be used downstream from construction sites, but sediment removal is difficult. They can be used to improve runoff water quality from roads, parking lots, residential, commercial and industrial areas. Typically, they are used in conjunction with other onsite BMPs.

<u>Design Considerations</u>

a. See Chapter 7 for design criteria for detention basins.

<u>Advantages</u>

a. Cost-effective for larger tributary watersheds.

b. Moderate to high removal rates of many urban pollutants.

c. Creates wildlife habitat.

d. Provides recreation, aesthetics, and open space areas.

e. More efficient sedimentation than dry basin, since outlet is above the basin bottom, leaving a 'dead zone' to trap sediment.

DRAFT

f. Infiltration and resultant recharge to ground water is minimal compared to infiltration type BMPs; therefore, the risk of direct introduction of contaminants to ground water is reduced.

g. May be more efficient over time due to increased vegetation providing enhanced nutrient and metals removal rates.

Disadvantages

a. Attract waterfowl, which may increase downstream nutrient loading and bacteria.

b. Inadequate base flow can cause very high concentrations of salts, nutrients, and algae through evaporation, resulting in significant downstream loadings from smaller events.

c. Possible low dissolved oxygen effluent, stream warming, tropic shifts, habitat destruction, and loss of upstream channels.

d. Large events or low dissolved oxygen content can cause mixing or re-suspension of deposited sediments, increasing turbidity and metals concentrations.

e. Very space/land intensive (high opportunity cost), represent a safety liability (drowning, harmful/toxic algal blooms, and high concentrations of pollutants.

f. Higher cost than conventional storm water detention.

g. Sediments must be removed regularly via expensive dredging practices to ensure maximal pollutant removal.

h. Floating litter, scum and algal blooms, odors, insects.

i. High potential for stratification and anoxic conditions.

j. Bottom of pool may need to be lined to maintain permanent pool in well-draining conditions.

k. Wet retention ponds have greater storage capacity requirements than dry extended dry detention basins, resulting in higher capital costs.

l. Large basins may require a dam safety permit.

DRAFT

<u>Maintenance/Monitoring/Enforcement Considerations</u>

a. Sediment to be removed when 20% of storage volume of the facility is filled (design storage volume must account for volume lost to sediment storage).

b. No woody vegetation shall be allowed on the embankment without special design provisions.

c. Native wetland plants should be maintained along the perimeter of the pond.

d. Other vegetation over 18 inches high shall be cut unless it is part of planned landscaping.

e. Debris shall be removed from blocking inlet and outlet structures and from areas of potential clogging.

f. The control shall be kept structurally sound, free from erosion, and functioning as designed.

g. Bulkheads are prohibited in order to minimize safety risks and to ensure that littoral vegetation will be able to become established.

h. Control of scum and algal blooms, odors, insects.

i. The site shall be inspected and debris removed after every major storm.

j. Funds must be budgeted for routine and non-routine maintenance, particularly considering the high cost of sediment removal. For this reason, public rather than private maintenance is preferred.

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Figure 11-2. Retention (Wet) Pond

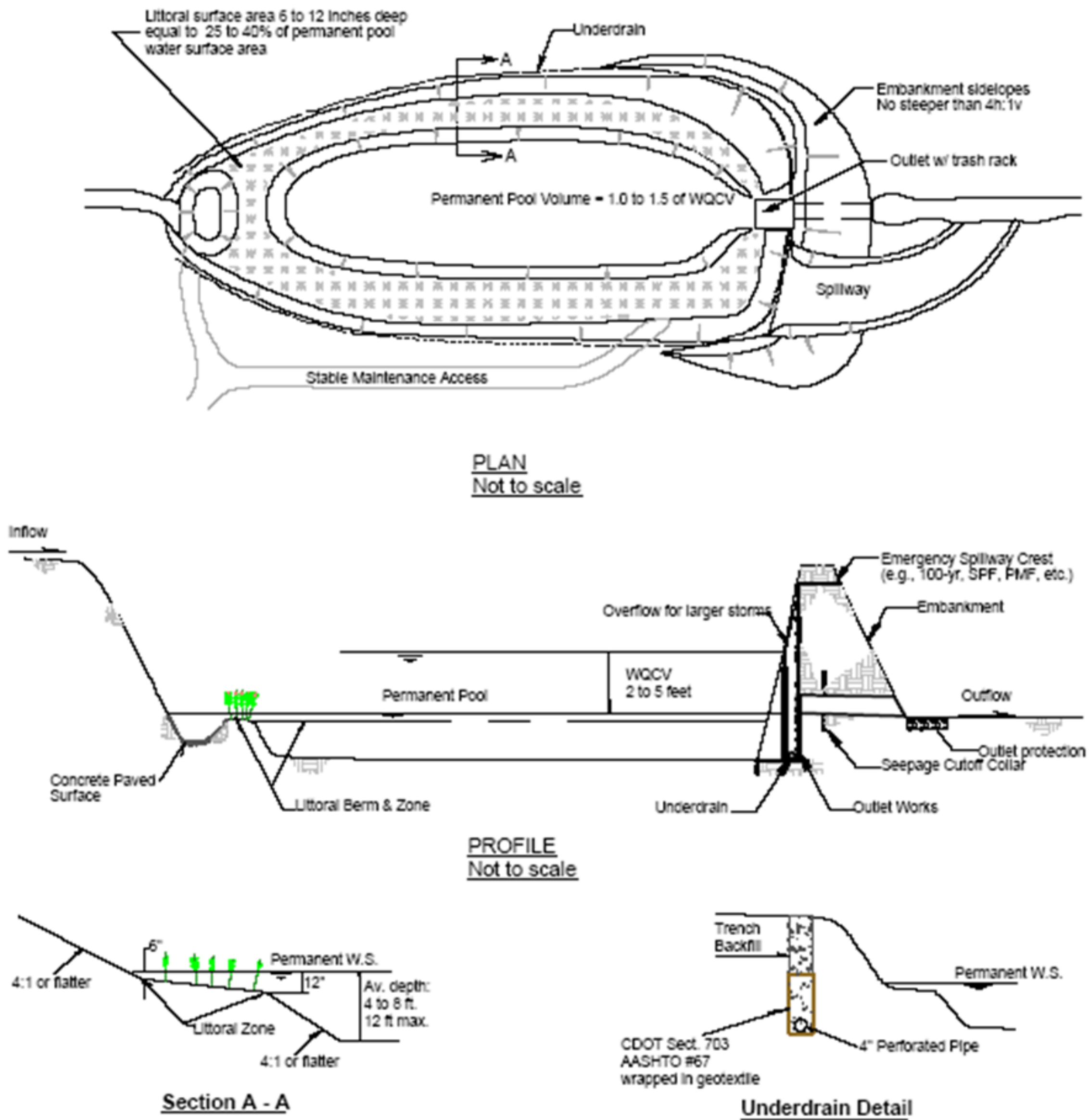
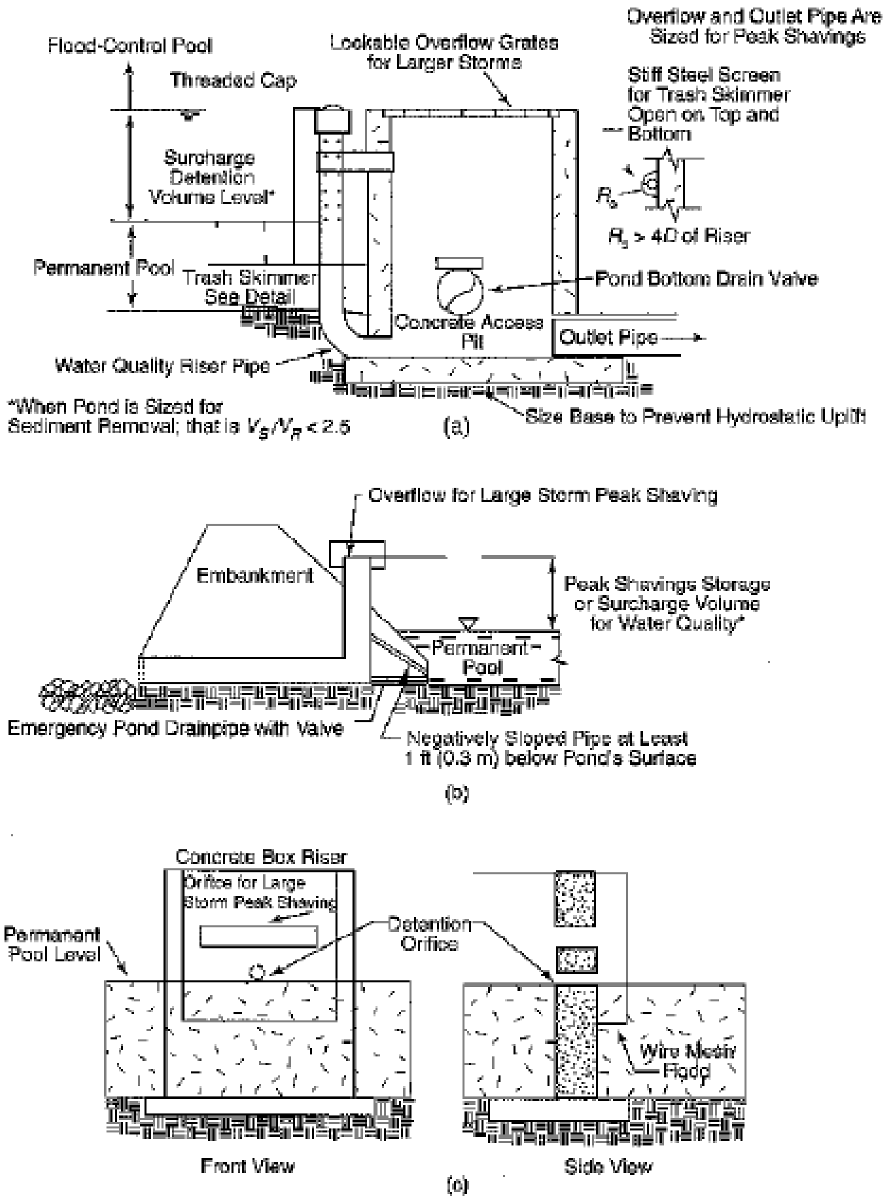


Figure 11-3. Schematic Diagrams of Wet Pond Outlets



11.4.4.3 Constructed Wetlands

Constructed wetlands can take the form of very shallow retention ponds or wetland-bottomed channels. A perennial base flow is needed to encourage the growth of wetland species such as rushes, willows, cattails, and reeds. The vegetation slows runoff and promotes settling and biological uptake. "Pocket" wetlands are typically under a tenth of an acre in size, serving developments of 10 acres or less. These are usually less reliable and efficient than larger wetlands (see Figures 11-4 and 11-5 for representative schematics).

Typical Applications

a. Wetland basins can be used as a follow-up BMP in a watershed or as an onsite facility if the owner can provide sufficient water. Flood control measures may be instituted above the wetland basin.

b. Retrofitting to established developments may be very difficult due to areal requirements.

c. Arid climates or high evapotranspiration rates can make maintenance of the required base flow difficult. Also, short growing seasons may inhibit vegetative growth and propagation.

d. Wetland bottom channels can be used in two ways. First, a wetland can be established in a man-made channel and can act as both a conveyance facility and a water quality enhancement facility. The second, and possibly more effective option, is to locate the channel downstream of a storm water detention facility that will remove much of the sediment load; the channel then provides better quality water to the receiving water body. The detention facility shall have at least a 24 hour drain period for the design storm.

Design Considerations

a. See Chapter 7 for design criteria for detention basins.

Advantages

a. Aesthetics, wildlife habitat, erosion control, pollutant removal.

Disadvantages

a. Possible stream warming, natural wetlands alteration.

b. Salts and scum may accumulate and be flushed out with a major storm event.

DRAFT

c. Possible breeding ground for pests and mosquitoes.

d. Effectiveness at removing nitrogen and some forms of phosphates is questionable.

e. Need for periodic sediment removal to maintain proper distribution of growth zones and water movement.

<u>Maintenance/Monitoring/Enforcement Considerations</u>

a. Difficult to determine, but with proper design and maintenance the wetland should perform well for an indefinite period of time.

b. Proper depth and spatial distribution of growth zones must be maintained.

c. Remove unwanted vegetation, debris and litter, accumulated sediment and organic muck.

d. Maintenance is generally greatest during the first three years in order to establish vegetation.

DRAFT

Figure 11-4. Wetland Pond (Plan and Profile)

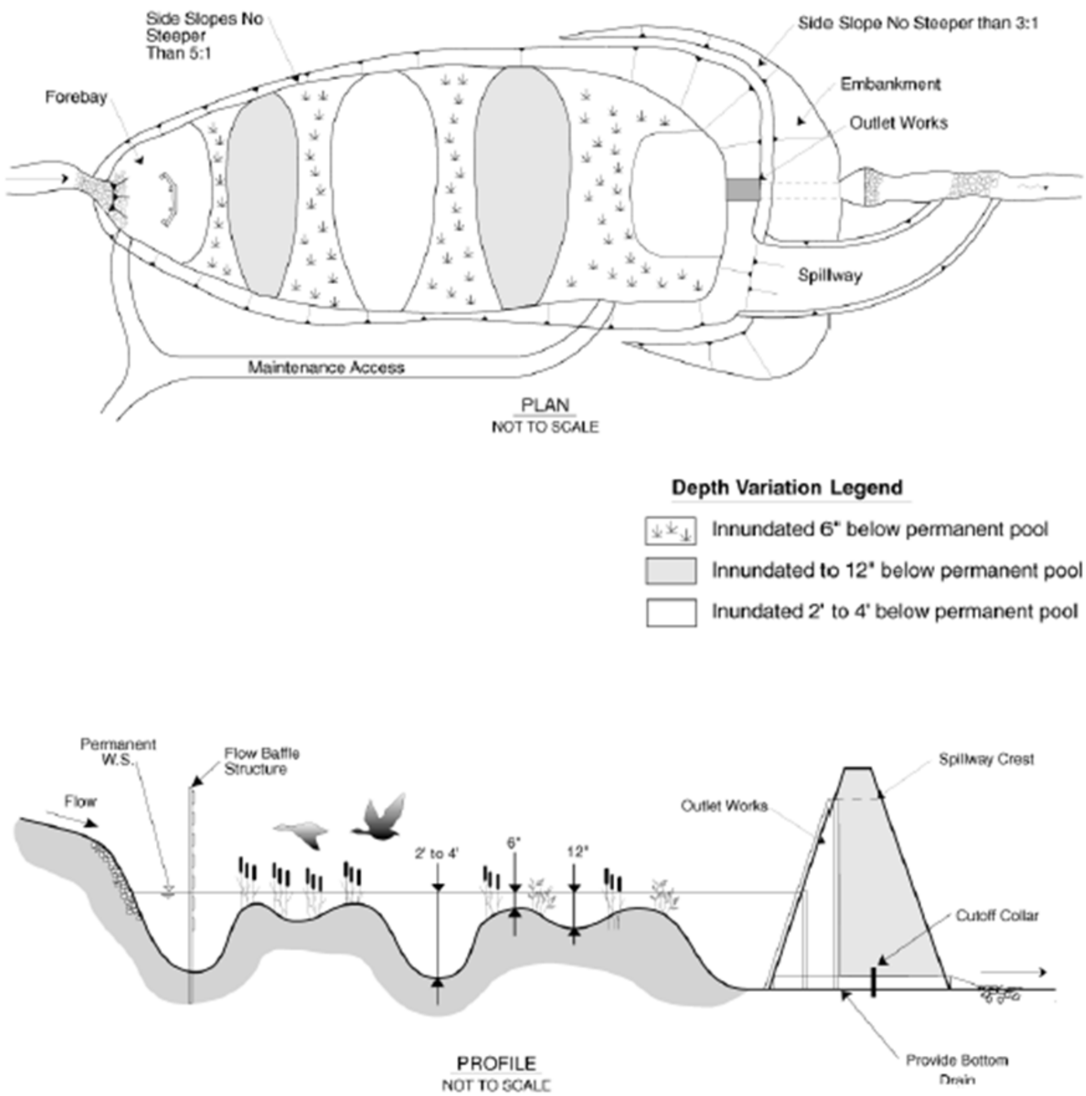
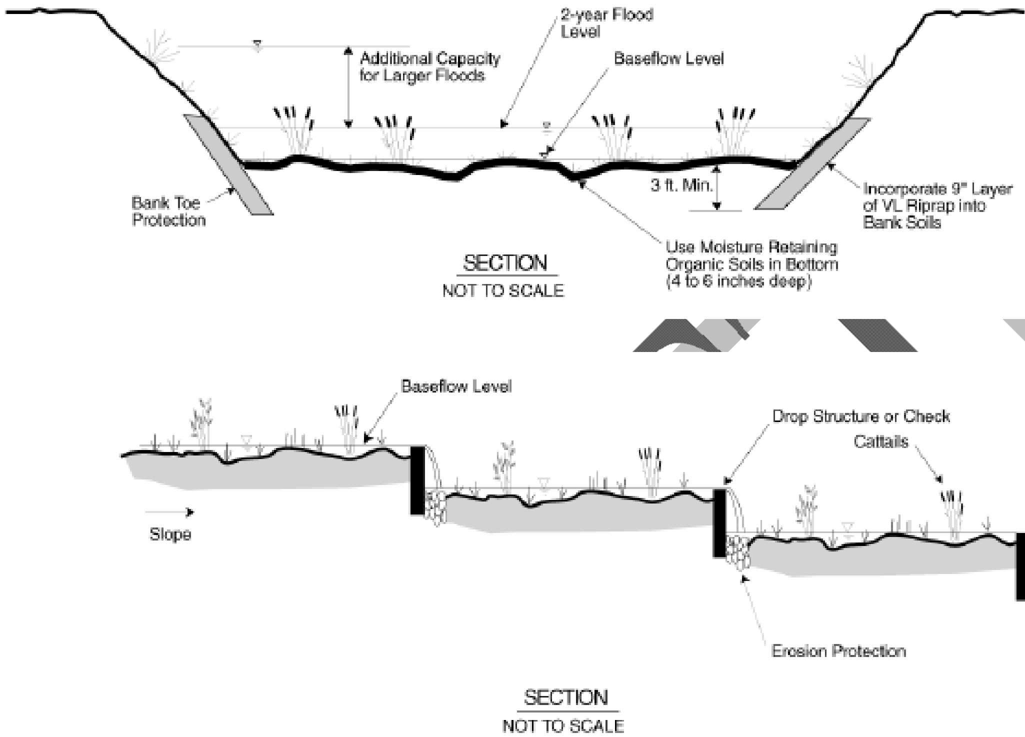


Figure 11-5. Wetland Channel (Section and Profile)



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11.4.4.4 Grassed Swales

Grassed swales are densely vegetated drainageways with low-pitched side slopes that collect and slowly convey runoff. The emphasis is on slow, shallow flow that encourages sedimentation and discourages erosion. They are set lower than the surrounding ground level, allowing runoff to enter the swales over grassy, shallow banks. Check dams may be used in conjunction with the swales to further slow the runoff. If base flow is present, wetland vegetation may also develop (see Figures 11-6 and 11-7 for a representative schematic).

<u>Typical Applications</u>

a. Swales are often used to collect overland flow from impervious areas such as parking lots, buildings and roadways, as well as semi-pervious areas such as grass filter strips and residential yards. A follow-up BMP will be required to enhance water quality.

<u>Design Considerations</u>

a. Generally well adapted for sites with ground slopes up to 3 or 4 percent, and not over 6 percent. The longitudinal slope of the swale shall be less than 1 percent.

b. Limited to runoff velocities less than 2.5 ft/s.

c. Maximum design flow depth should be 3 feet.

d. Swale cross-section shall have side slopes of 4:1 or flatter.

e. Underlying soils shall have a high permeability.

f. Swale area shall be tilled before grass cover is established.

g. Dense cover of a water tolerant, erosion resistant grass shall be established over swale area.

h. As a BMP, grassed swales are best suited to residential or institutional areas where percentage of impervious area is relatively small.

i. Check dams can be installed in swales to promote additional infiltration. Recommended method is to sink a railroad tie halfway into the swale. Riprap stone shall be placed on the downstream side to prevent erosion.

Advantages

a. Aesthetics.

b. Effective in reducing runoff in small storm events where other BMPs are less effective.

c. Can be used to limit the extent of directly connected impervious areas.

Disadvantages

a. Potential for soggy yards, mosquito breeding, and more right-of-way requirement than for equivalent storm sewers.

b. Particularly with small storm events, the primary removal mechanism is infiltration; in areas of high ground water vulnerability, this may not be a good option.

c. Effectiveness is limited by infiltration capacity of soils; conversely, well-draining soils may direct polluted runoff directly to ground water.

Maintenance/Monitoring/Enforcement Considerations

a. Dependent on proper design and maintenance.

b. Routine maintenance: grass must be mowed, some litter removal, sediment removal to maintain channel flow capacity.

c. Non-routine maintenance: replacement of damaged grass and/or check dams.

d. Maintenance must be included in the budget to insure proper operation.

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Figure 11-6. Example Schematic of a Grassy Swale with a Check Dam

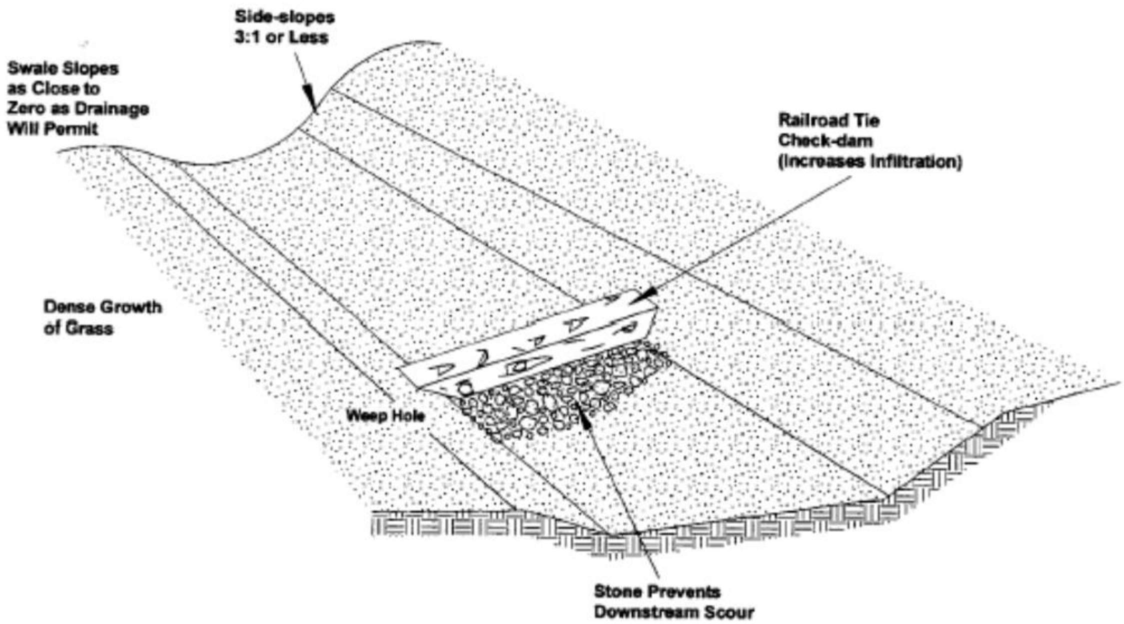

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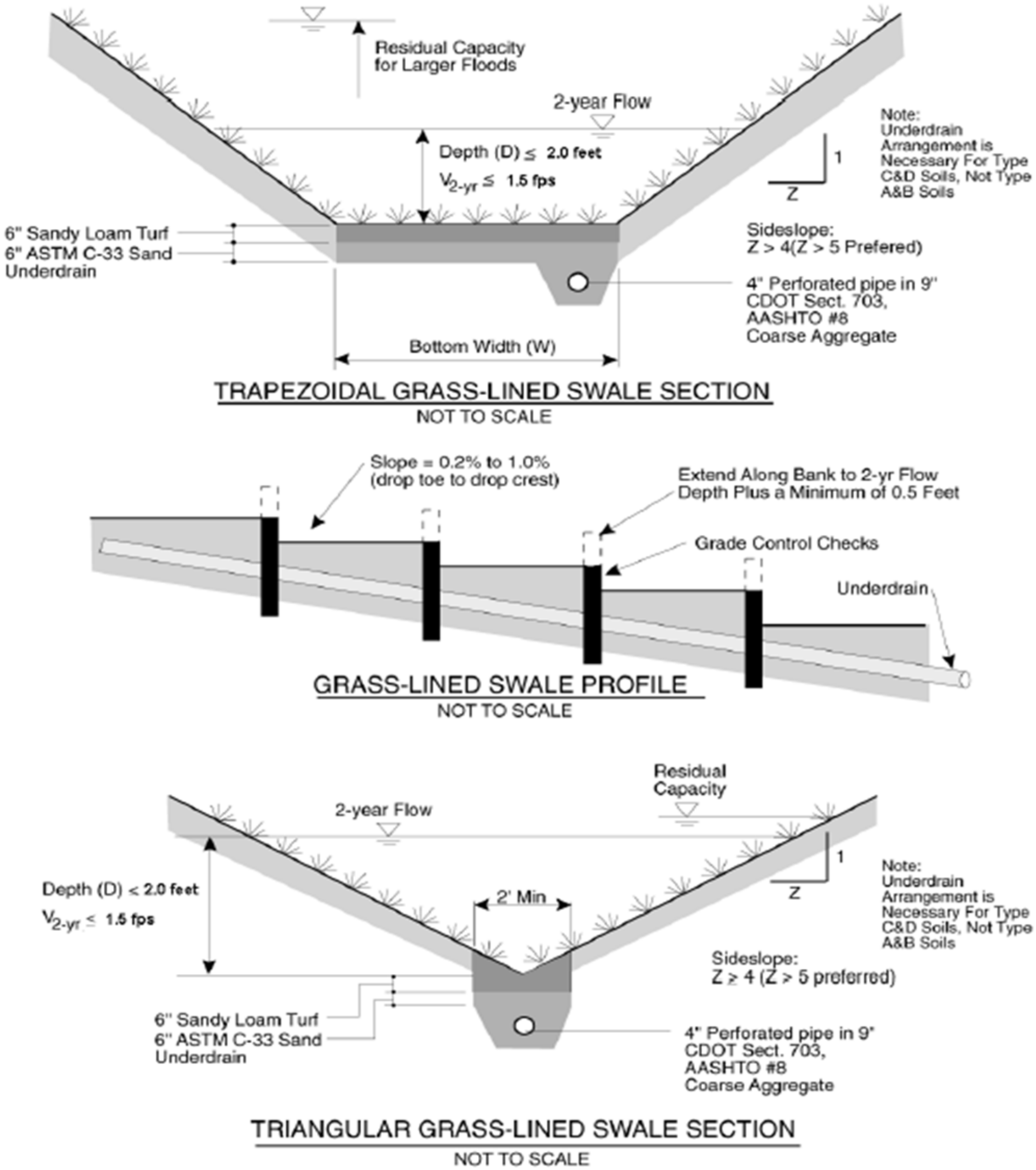
Figure 11-7. Profile and Sections of a Grass-Lined Swale



11.4.4.5 Filter Strips and Flow Spreaders

Filter strips are vegetated areas designed to accept sheet flow provided by flow spreaders, which accept flow from an upstream development. Vegetation may take the form of grasses, meadows, forests, etc. The primary mechanisms for pollutant removal are filtration, infiltration, and settling (see Figures 11-8 through 11-10 for representative schematics).

Typical Applications

a. Filter strips can be used in residential and commercial sites, and adjacent to impervious areas. Effectiveness depends on evenly distributed sheet flow, and limited drainage area and runoff volume. For grass filter strips, the environment must support turf-forming grasses. They have limited effectiveness in pollutant removal, and follow-up structural BMPs will still be required.

Design Considerations

a. The proper function of the flow spreader is key to the performance of the filter strip. If flow is allowed to concentrate, the bulk of the filter strip will be ineffective for pollutant removal and flow reduction. This will also result in erosion over time.

b. Flow spreaders and filter strips shall be limited to drainage areas of 5 acres or less.

c. Channel grade for the last 20 feet of the dike or diversion entering the level spreader shall be less than or equal to 1% and designed to provide a smooth transition into spreader.

d. Grade of level spreader shall be 0%.

e. Depth of level spreader as measured from the lip shall be at least 6 inches.

f. Recommended length, width, and depth of flow spreaders are presented in Table 11-1:

Table 11-1. Recommended Flow Spreader Dimensions

Design Flow (cfs)	Entrance Width (ft)	Depth (ft)	End Width (ft)	Length (ft)
0 - 10	10	0.5	3	10
10 - 20	16	0.6	3	20
20 - 30	24	0.7	3	30

DRAFT

g. Level spreader lip shall be constructed on undisturbed soil (not fill material) to uniform height and zero grade over length of the spreader.

h. Released runoff to outlet shall be onto undisturbed stabilized areas in sheet flow and not allowed to re-concentrate below the structure.

i. Slope (S_o) of filter strip from level spreader shall not exceed 10 percent.

j. The design width of the filter strip (W_G) shall be the greater of the following:

$$W_G \geq 10\ feet, or\ W_G \geq 0.2L_1 \qquad \textbf{Equation 11-1}$$

where: L_l = the length of flow path of the sheet flow over the upstream impervious surface

k. Spreader lip shall be protected with erosion resistant material, such as fiberglass matting or a rigid non-erodible material for higher flows, to prevent erosion and allow vegetation to be established.

l. Wooded filter strips are preferred to gravel strips.

<u>Advantages</u>

a. Aesthetics of open, green space.

b. Low cost, since developments are typically required to have open space in their plans.

c. Grasses and shrubs or trees provide wildlife habitat.

<u>Disadvantages</u>

a. The primary flood-control mechanism is infiltration; in areas of high ground water vulnerability, this may not be a good option.

b. On unstable slopes, soils or vegetation, rills and gullies may develop that negate the usefulness of the strips.

c. Excessive pedestrian or vehicle traffic may damage the vegetation and soils structure. The planting of shrubs and trees can help eliminate both of these disadvantages.

d. Inadequate maintenance of vegetation may result in partially denuded soils with predictable results in erosion, runoff quality, and volume.

DRAFT

<u>Maintenance/Monitoring/Enforcement Considerations</u>

a. Routine maintenance consists of standard turf maintenance, and removal of sediment deposits and any projections or other irregularities which will impede normal flow.

b. Non- routine maintenance consists of turf replacement, soils replacement, and regrading.

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Figure 11-8. Perspective View of a Level Spreader.

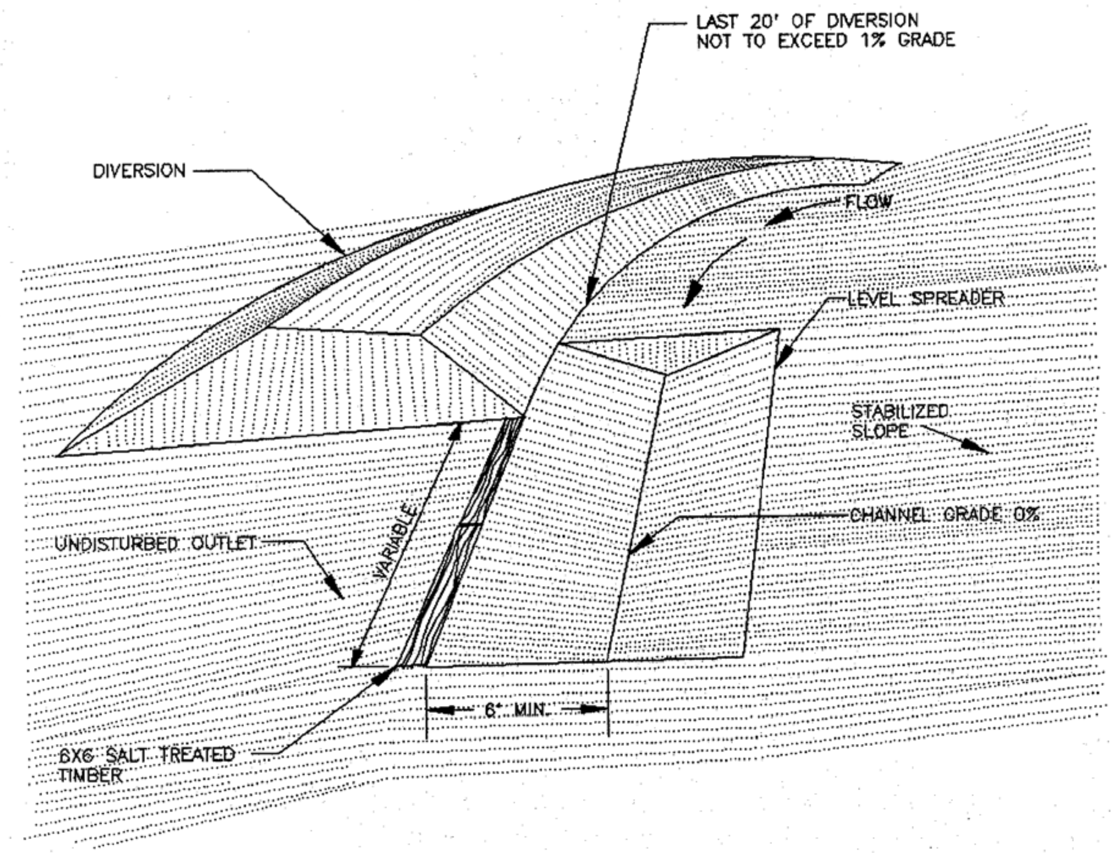

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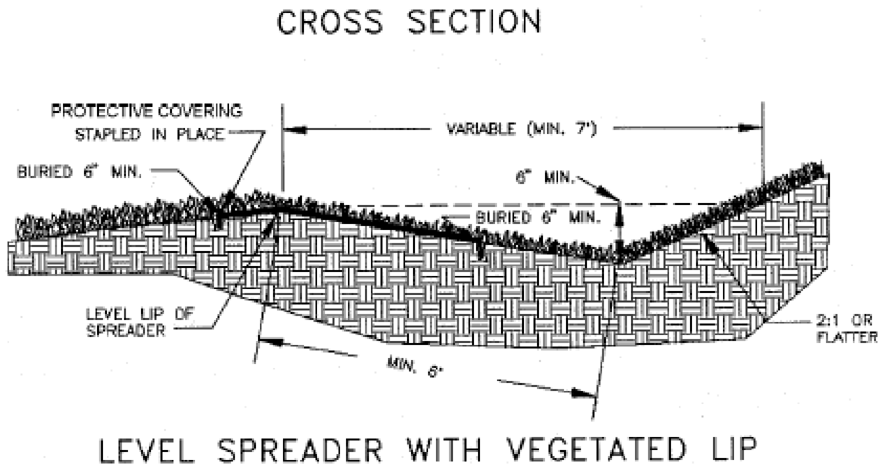
Figure 11-9. Cross-Section of a Level Spreader



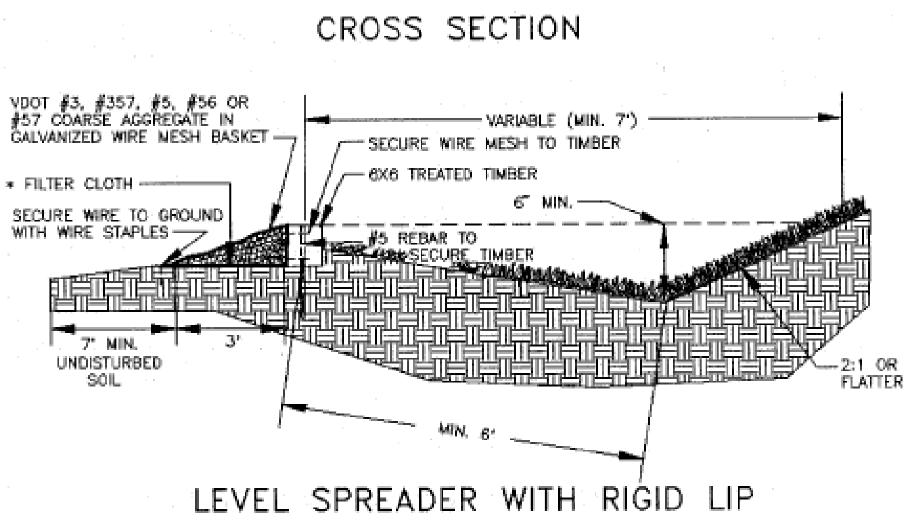
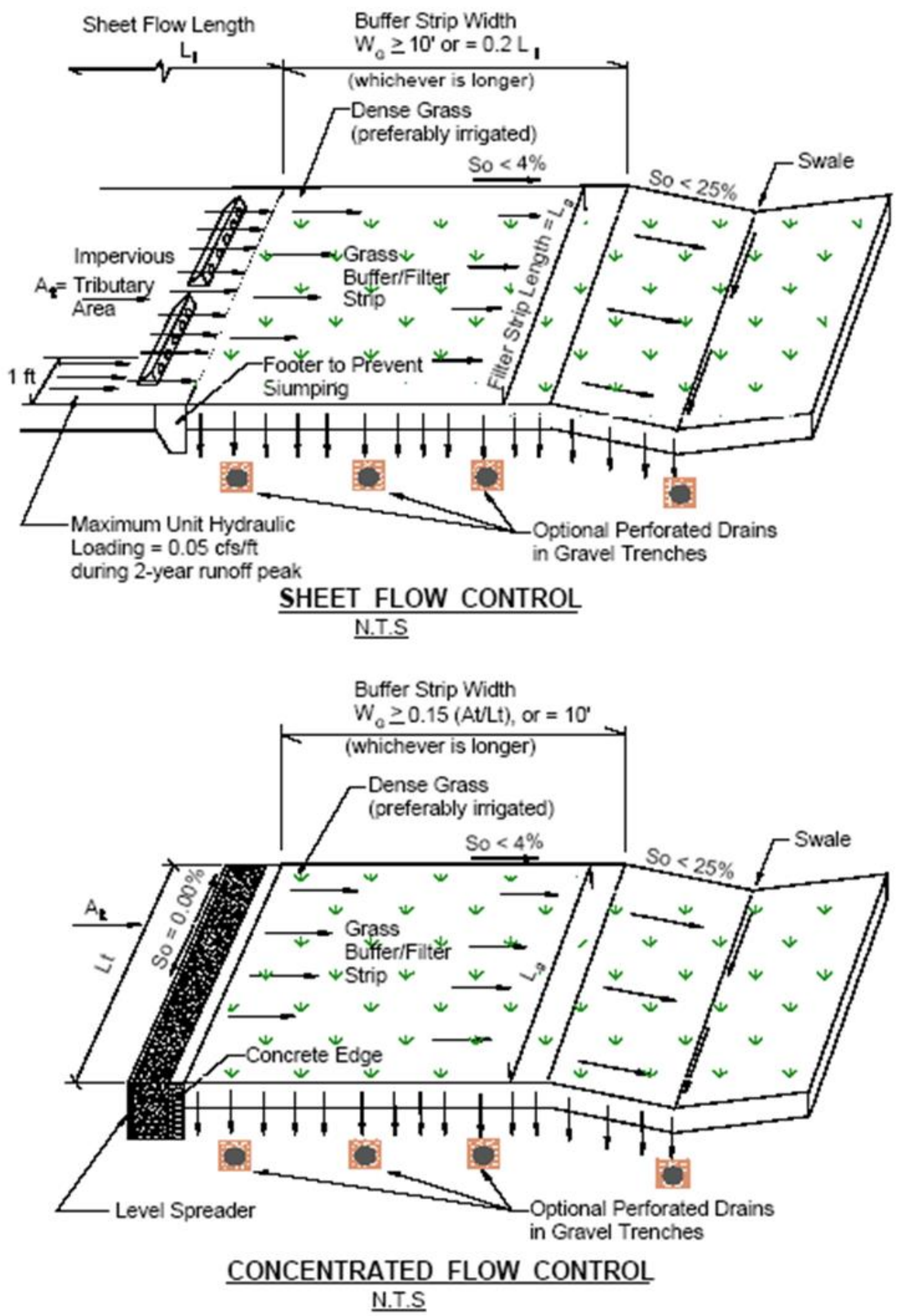


Figure 11-10. Onsite and Offsite Applications of Grass Filter Strips



11.4.4.6 Sand Filters

In its simplest form, a sand filter is a self-contained bed of sand into which the first flush of runoff is diverted. The water is filtered as it passes through the sand, much like a slow sand filtration system for drinking water supply. The effluent is typically collected with a perforated pipe and discharged to a stream or channel.

Sand filters are often placed at the outlet of detention basins to improve effluent water quality. Enhanced sand filters use layers of peat, limestone, and/or topsoil to improve removal rates. Sand trench systems, such as the Washington D.C. and Austin sand filters, are used to treat parking lot runoff (see Figures 11-11 and 11-12 for a representative schematics).

A new modification of the sand filter concept is the biofiltration pond. Using a media which has a cation exchange capacity of at least 10 milli-equivalents per 100 grams will improve metals capture. Although sand is still the predominant media of choice, clays and other compounds may be included to attain high pollutant removal rates while still providing ample drainage for the design storm event. This can typically be accomplished using a gradation of filter media, decreasing in size with depth.

<u>Typical Applications</u>

a. Sand filters have been successfully used in diverse applications for small (less than 10 acres) tributary areas.

b. Recommended for "ultra-urban" areas where area is limited, and runoff is poor quality; not recommended for new construction sites.

c. Most sand filters are limited to an impervious tributary area of 5 to 10 acres. Follow-up sand filters, placed at the outlet of detention basins, may treat tributary areas in excess of 100 acres.

<u>Design Considerations</u>

a. Inlet structure shall be designed to spread the flow uniformly across the surface of the filter media.

b. Riprap or other dissipation devices shall be installed to prevent gouging of the sand media and to promote uniform flow.

c. Final sand bed depth shall be at least 18 inches.

d. Underdrain pipes shall consist of main collector pipes and perforated lateral branch pipes.

DRAFT

e. The underdrain pipes shall be reinforced to withstand the weight of the overburden.

f. Internal diameters of lateral branch pipes shall be 4 inches or greater and perforations shall be $^{3}/_{8}$ inch.

g. Maximum spacing between rows of perforations shall not exceed 6 inches.

h. All pipes shall be schedule 40 polyvinyl chloride or greater strength.

i. Minimum grad of pipes shall be 1% slope.

j. Access for cleaning all underdrain pipes shall be provided.

k. A pre-settling basin and/or biofiltration swale is recommended to pretreat runoff discharging to the sand filter.

l. A maximum spacing of 10 feet between lateral underdrain pipes is recommended.

m. The primary purpose of the sand filter is to improve storm water quality; they have a limited ability to reduce peak flows.

Advantages

a. Since infiltration is not a significant mechanism, ground water protection is maximized.

b. This BMP has a proven performance record in a variety of applications.

c. Since the removed sand has been demonstrated to be non-toxic, and there is no evidence that re-suspension of contaminated sediment is a problem, there is little concern for environmental problems with this BMP.

Disadvantages

a. Larger sand filters with no vegetative cover may be unattractive; the surface can be extremely unattractive, and some have caused odor problems.

b. Sand filters are primarily for storm water quality mitigation, not quantity or peak flow mitigation.

DRAFT

Maintenance/Monitoring/Enforcement Considerations

a. Routine maintenance includes debris removal and scraping of the upper sand layer. This is mostly manual work.

b. Non- routine maintenance includes re-sanding (replacement of the sand) after enough sand has been removed that significant breakthrough occurs. In designs that include a sedimentation basin, it must be cleaned out when the basin loses its holding capacity.

DRAFT

Figure 11-11. Washington D.C. Sand Filter Design

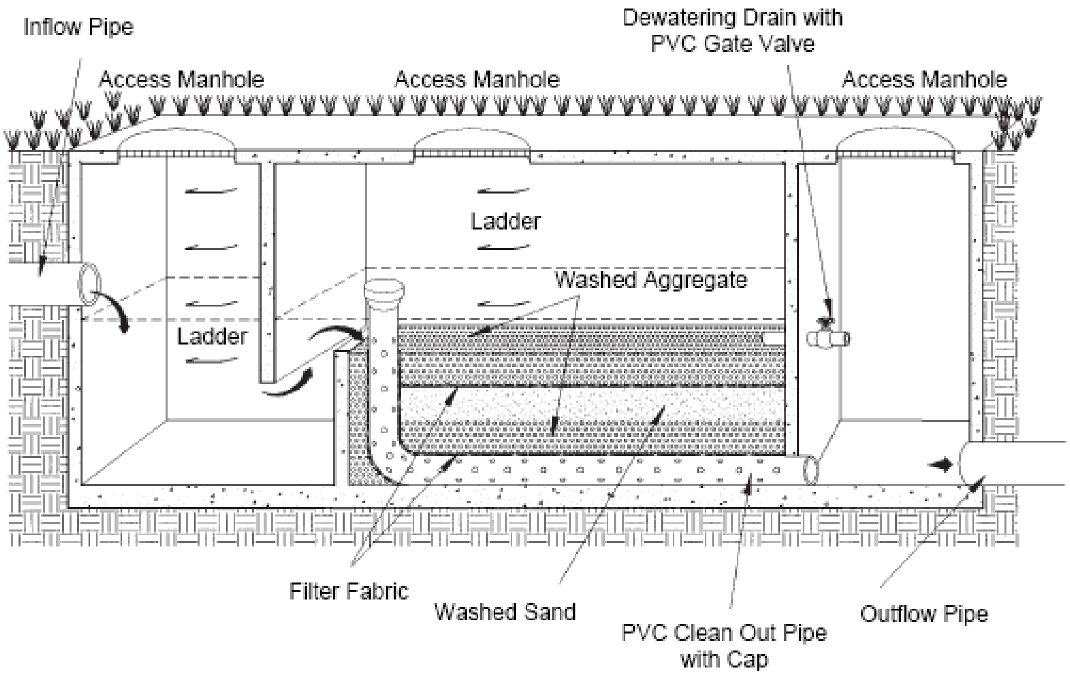
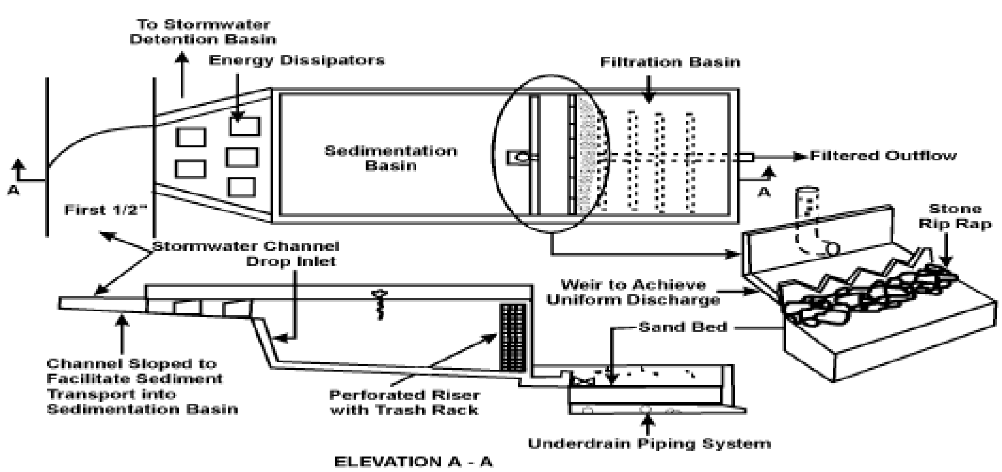


Figure 11-12. Austin Sand Filter Design



11.4.4.7 Infiltration Trenches

Infiltration trenches can be generally described as a part of an open ditch that encourages rapid infiltration of runoff to the ground water through the placement of materials with high hydraulic conductivities. The trench is basically an excavated area within the ditch into which clean gravels are placed. The ditch shall provide for slow flow rates to allow settling of suspended solids as well as the opportunity for substantial infiltration of the total intercepted flow (see Figure 11-13 for a representative schematic).

<u>Typical Applications</u>

a. As an infiltration type BMP, use shall be limited to those areas where ground water levels are well below the bottom of the trench and there is significant retention time in the soils before reaching ground water.

b. Infiltration trenches work well for residential lots, commercial areas, parking lots, and open space areas.

<u>Design Considerations</u>

a. Use in drainage areas less than 15 acres.

b. Commonly, infiltration trenches are sized to intercept and dispose of runoff from a specific design storm (typically 2-year storms).

c. Soils that are suitable for infiltration systems are silt loam, loam, sandy loam, loamy sand, and sand.

d. Soils that have a 30 percent or greater clay content are not suitable for infiltration trenches).

e. The soil infiltration rate shall be between 0.5 and 2.4 inches per hour.

f. The use of infiltration systems on fill is not allowed due to the possibility of creating an unstable subgrade.

g. A minimum of 3 feet between the bottom of the infiltration trench and the groundwater table is recommended.

h. Site slope shall be less than 20 percent and trench shall be horizontal.

i. The proximity of building foundations shall be at least 10 feet upgrade.

j. A minimum of 100 feet shall be maintained from water supply wells when the runoff is from industrial or commercial areas.

DRAFT

k. Water quality infiltration trenches shall be preceded by a pretreatment BMP.

l. The aggregate material for the trench shall consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches.

m. The aggregate shall be graded such that there will be few aggregates smaller than the selected size. For design purposes, void space for these aggregates may be assumed to be in the range of 30 to 40 percent.

n. The aggregate shall be completely surrounded with an engineering filter fabric. If the trench has an aggregate surface, filter fabric shall surround all aggregate fill material except for the top one foot.

o. Bypass larger flows.

p. To reduce clogging of the trench with sediments, a sump pit or a filter strip and flow spreader shall be used to treat water entering the ditch.

q. Infiltration systems shall not be constructed until all construction areas draining to them are fully stabilized.

r. An analysis shall be made to determine any possible adverse effects of seepage zones when there are nearby building foundations, basements, roads, parking lots, or sloping sites.

<u>Advantages</u>

a. The combination of water conveyance, runoff reduction, lowering of peak flows, and pollutant removal make this an effective BMP.

<u>Disadvantages</u>

a. The use of infiltration as the primary pollutant reduction mechanism may increase ground water contamination by highly soluble contaminants in fast-draining soils and/or high water level conditions.

b. If a trench becomes clogged with sediments, it simply stops functioning. The gravel must be removed and replaced with clean gravel, and it may be necessary to remove soils lining the trench which have also become clogged.

c. If the trench becomes fully clogged, complete rehabilitation may cost as much as initial construction; if funding is private, the trench may go unrepaired.

DRAFT

Maintenance/Monitoring/Enforcement Considerations

a. If non-routine maintenance is performed correctly, there should be little degradation in performance.

b. Routine maintenance includes debris and litter removal and control of overgrown vegetation.

c. Non-routine maintenance involves a clogged trench which requires complete removal and replacement of the gravel as well as surrounding clogged native soils. This can be greatly reduced by proper design and routine maintenance.

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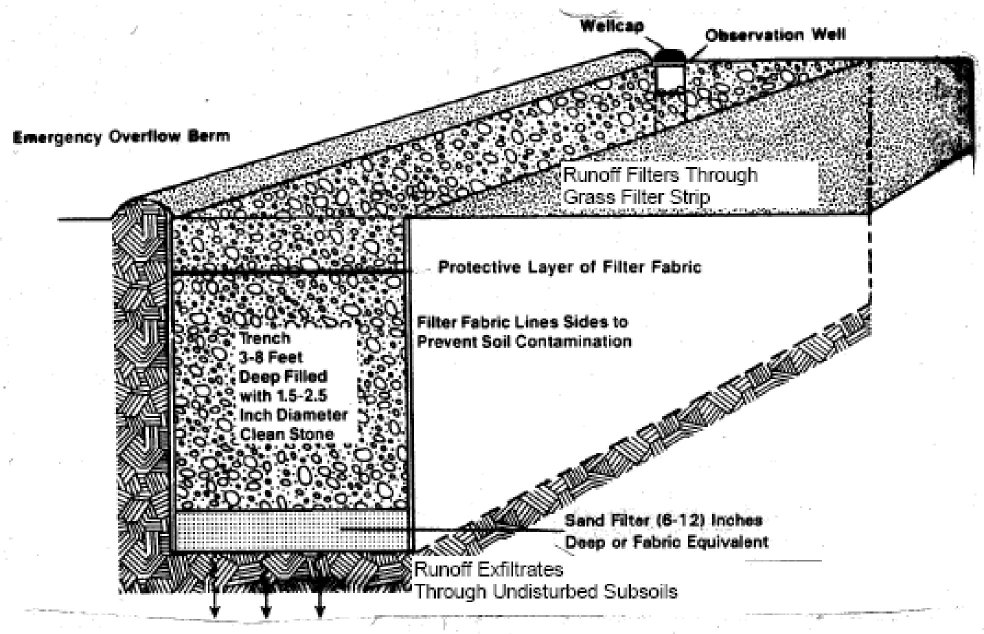
Figure 11-13. Schematic of an Infiltration Trench



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11.4.4.8 Porous Pavement

There are two forms of porous pavement: modular block, which is made porous through its structure, and poured-in-place concrete or asphalt which is porous due to the mix of the materials (see Figures 11-14 and 11-15 for representative schematics).

Modular block porous pavement consists of perforated concrete slab units underlain with gravel. The surface perforations are filled with coarse sand or sandy turf. It is used in low traffic areas to accommodate vehicles while facilitating storm water runoff at the source. It should be placed in a concrete grid that restricts horizontal movement of infiltrated water through the underlying gravels.

Poured-in-place porous concrete or asphalt is generally placed over a substantial layer of granular base. The pavement is similar to conventional materials, except for the elimination of sand and fine particles from the mix.

If infiltration to ground water is not desired, a liner may be used along with perforated pipe and a flow regulator to slowly drain the water away over a 6 to 12 hour period.

Porous pavement should be constructed in accordance with manufacturer's recommendations, as well as the requirements of this chapter. Structural requirements for pavement must also be taken into account.

<u>Typical Applications</u>

a. Modular block pavement is applicable to low traffic zones such as parking lots, service roads, emergency and utility access lanes, and permeable upper soils with ground water no less than 4 feet from the gravel bedding.

b. This is exclusively an on-site BMP that shall never be used for treating water with high sediment loads. This is particularly true for porous concrete or asphalt, which are primarily designed to remove pollutants deposited on the pavements from the atmosphere.

<u>Design Considerations</u>

a. Either form of porous pavement must be limited to low traffic areas with limited deposition of clays and fine particles which could clog the pavement.

b. As infiltration is the main mechanism of pollutant removal, areas with high ground water vulnerability may not be good choices for this BMP.

DRAFT

c. Large soil surface areas are needed for maximum exfiltration and pollutant removal.

d. Soil infiltration rate shall be greater than 0.27 inches per hour and clay content less than 30 percent.

e. Only feasible on sites with gentle slopes (less than 5%).

f. Design infiltration rate shall be equal to ½ of the infiltration rate determined from soil textural analysis.

g. Minimum of 3 feet between stone reservoir level and seasonally high-water table.

h. Shall not be constructed over fill soils.

i. Vegetative strip or diversion berm required to protect pavement area from off-site runoff before, during, and after construction.

j. If porous pavement areas receive runoff from off-site areas, a pretreatment facility shall be constructed to remove oil, grit, and sediments before entering the porous pavement.

k. Dry subgrade shall be covered with filter fabric on bottom and sides.

l. Poured-in-place porous pavement section shall consist of 4 layers as shown on Figure 11-15.

m. Stone shall be clean, washed, and meet roadway standards.

n. Reservoir base course shall consist of 1 to 3 inches crushed stone aggregate compacted lightly at the depth required to achieve design storage.

o. Filter courses shall be ½ inch crushed stone aggregate at a 1 to 2 inch depth.

p. Surface course shall be laid in 1 lift at the design depth with compaction done while the surface is cool enough to resist a 10-ton roller. Only 1 or 2 passes are required.

q. After final rolling, no vehicular traffic shall be permitted on pavement until cooling and hardening - a minimum of 1 day.

r. Stone reservoir shall be designed to completely drain within a maximum of 2 to 3 days after design storm event, which is typically the 6-mo, 24-hr duration.

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s. The porous pavement site shall be posted with signs indicating the nature of the surface and warning against resurfacing, using abrasive equipment, and parking heavy equipment.

t. An observation well shall be installed on the downslope end of the porous pavement area to monitor runoff clearance rates. The observation well shall consist of perforated PVC pipe, 4 to 6 inches in diameter, constructed flush with the ground. The top of the observation well shall be capped to discourage vandalism and tampering.

u. Limited to sites between ¼ acre and 10 acres.

v. Shall not be constructed near groundwater drinking supplies.

w. Heavy equipment shall be prevented from compacting the underlying soils before and during construction.

x. Sand or ash shall not be applied to porous pavement.

<u>Advantages</u>

a. Low maintenance for modular block pavement.

b. Slows and reduces runoff, reducing the need for expensive detention facilities.

<u>Disadvantages</u>

a. Fast draining soils can result in ground water pollution from soluble metals and other pollutants.

b. Risk can vary from very minor to great, depending on how well the system is functioning.

c. Large silt and sand loads (e.g. from construction sites) may accelerate the clogging of the pavement pores, requiring expensive removal of sediments.

d. Porous concrete or asphalt tends to seal in 1-3 years unless vacuum cleaning is done frequently; even then, it will eventually seal. The need for vacuum cleaning makes this option more expensive for routine maintenance.

Maintenance/Monitoring/Enforcement Considerations

a. All porous pavement designs will degrade in performance over time, with careful maintenance only incrementally increasing its operational lifespan.

b. Maintenance is minimal for modular block except when the surface becomes clogged. This will require expensive non-routine maintenance in the form of removing the blocks and the underlying clogged gravels. Routine (quarterly) vacuum sweeping and high-pressure water washing of porous asphalt is required to prevent clogging. Non-routine maintenance consists of complete replacement and may be required in as little as one year's time.

c. When turf is used with modular block, lawn care maintenance is needed.

d. Spot clogging of the porous pavement layer can be relieved by drilling ¼ inch holes through the porous asphalt layer every few feet.

e. The obvious limitation is the need for expensive non-routine repairs or replacement. If privately owned, this expense may preclude necessary work. If publicly owned, there may be insufficient funds budgeted for maintenance.

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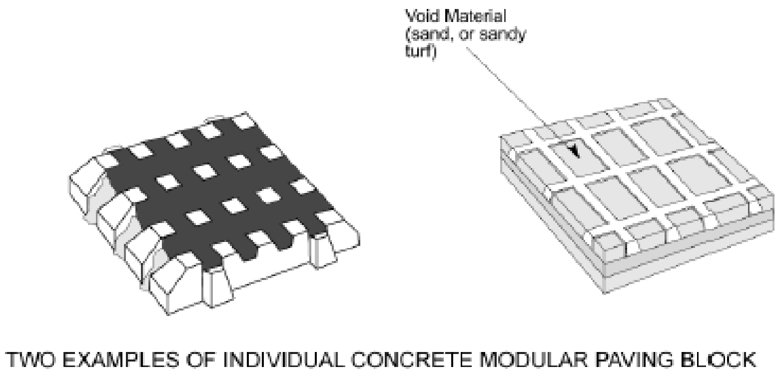
Figure 11-14. Design Schematic for Modular Block Porous Pavement



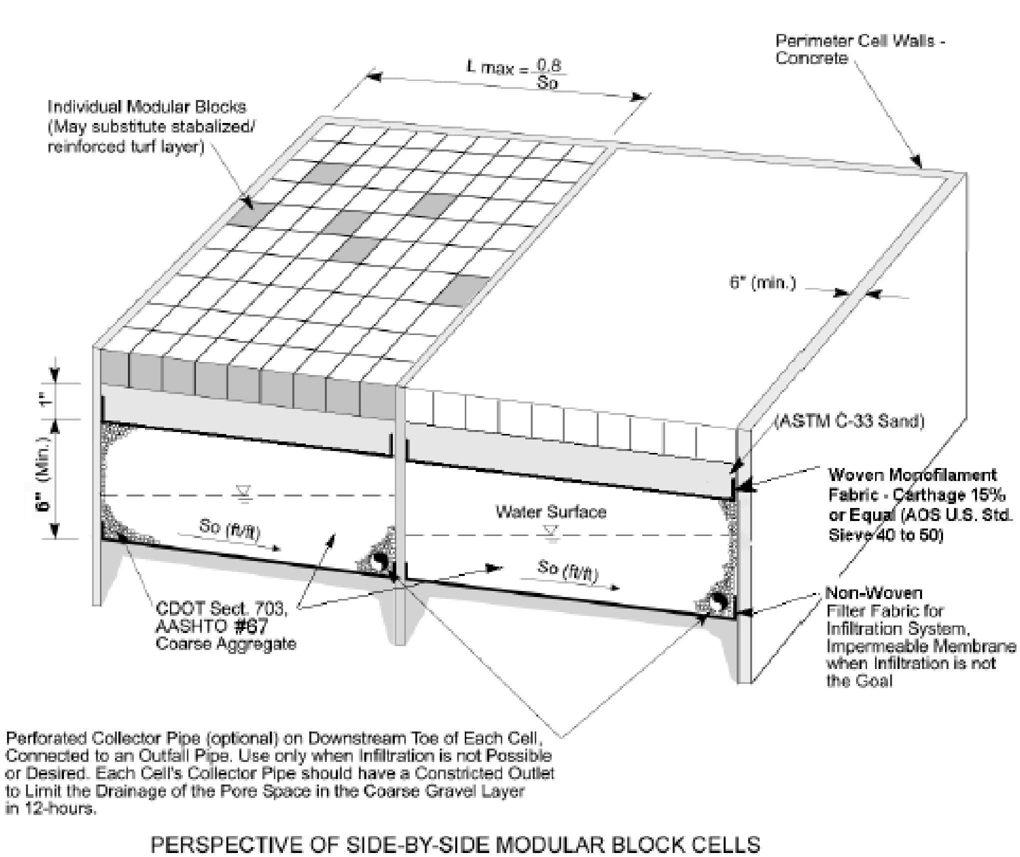
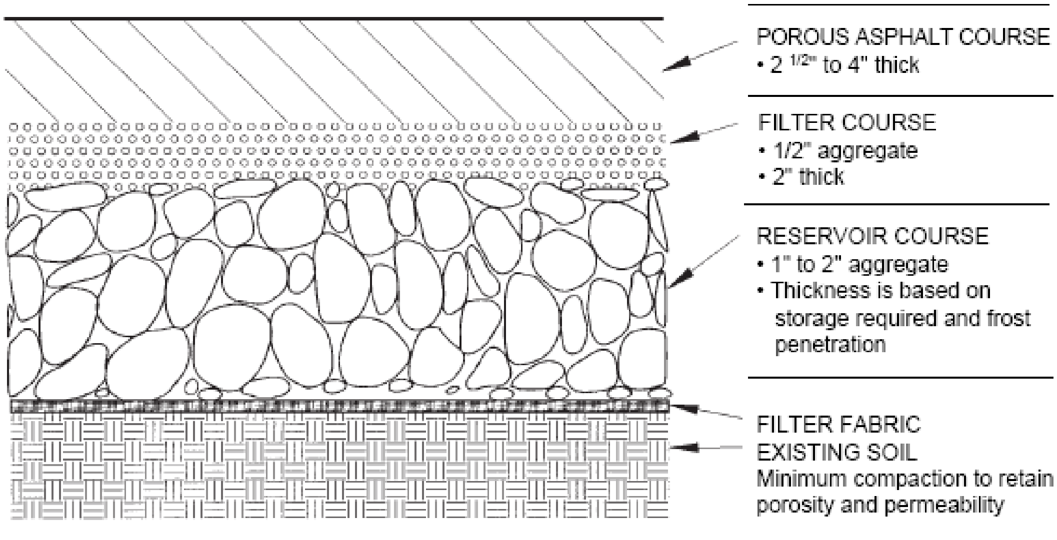


Figure 11-15. Porous Asphalt Pavement (Typical Section)



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11.4.4.9 Oil/Grit Separators

Also known as a water quality inlet, an oil and grit separator is a three-stage underground retention system designed to remove heavy particulates and hydrocarbons from runoff. The first chamber allows for sedimentation; the second chamber has an inverted elbow for an outlet, such that oil is trapped at the surface; and the third chamber directs the water out (see Figure 11-16 for a representative schematic).

Separators should be constructed in accordance with manufacturer's recommendations, as well as the requirements of this chapter.

<u>Typical Applications</u>

a. The most effective use of this BMP is in capturing runoff from small, high density sites (such as gas stations, parking lots, fast food restaurants, industrial loading facilities, and sections of industrial plants) where high concentrations of oils in runoff are expected.

b. Oil/Grit separators are most frequently used in highly urbanized areas where other BMPs cannot be used due to space limitations.

<u>Design Considerations</u>

a. Tributary area is usually limited to two acres or less of mostly impervious surfaces. This is primarily a water quality rather than quantity mitigation BMP.

b. Separator shall be structurally sound and designed for acceptable traffic loadings where subject to traffic loadings.

c. Separators shall be an off-line configuration, designed to capture only the first flush (½- inch) of runoff from the drainage areas. Upstream isolation/diversion structures can be used to divert the water to the off-line structure.

d. Separator shall be designed to be water tight to prevent possible ground water contamination.

e. Volume of separator shall be at least 400 cubic feet per acre tributary to the facility (first two chambers).

f. Forebay or first chamber shall be designed to collect floatable and larger settleable solids.

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g. Oil absorbent pads, oil skimmers, or other ap proved methods for removing accumulated oil shall be provided.

h. Separator pool shall be at least 4 feet deep.

i. Manholes shall be provided to each chamber to provide access for cleaning.

j. Separator shall be located close to the source before pollutants are conveyed to storm sewers or other BMPs.

k. Provide perforated covers as trash racks on orifices or pipes leading from first to second chamber.

<u>Advantages</u>

a. Can be used in highly urbanized areas where other BMPs cannot be used.

b. Trapping of floatable trash and debris and possible reduction of hydrocarbon loadings from impervious areas.

c. They do not rely on infiltration, so direct input of runoff into the ground water is unlikely.

<u>Disadvantages</u>

a. This BMP was originally designed for industrial applications, rather than urban storm water applications. When transformed as a storm water BMP, two problems arise: (1) an expectation of removal of pollutants other than oil and grit is created; and (2) widely varied flows can overwhelm and make ineffective a BMP that was designed for steady low flows and not fluctuating high flows.

b. Trapped sediments contain hydrocarbon by-products, and the residuals may be considered too toxic for conventional landfill disposal.

c. Large storm events can cause resuspension of trapped solids, resulting in a pulse of very poor-quality effluent.

d. The lack of a practical disposal method for the toxic sediments results in improper maintenance that causes failure of the system.

e. Pollutant removal performance likely drops off very quickly after a few months.

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Maintenance/Monitoring/Enforcement Considerations

a. The facility shall be checked weekly by the owner.

b. The facility shall be completely inspected and cleaned out at least twice a year to maintain the pollutant removal capabilities.

c. Sediment shall be cleaned out with a vacuum truck.

d. Oil absorbent pads, if used, are to be replaced as needed, but shall always be replaced after cleaning.

e. The effluent shutoff valve is to be closed during cleaning operations.

f. Waste oil and residuals must be disposed of in accordance with current Texas Water Code and/or Health Department requirements.

g. Any standing water removed shall be replaced with clean water to prevent oil carry-over through the outlet weir or orifice.

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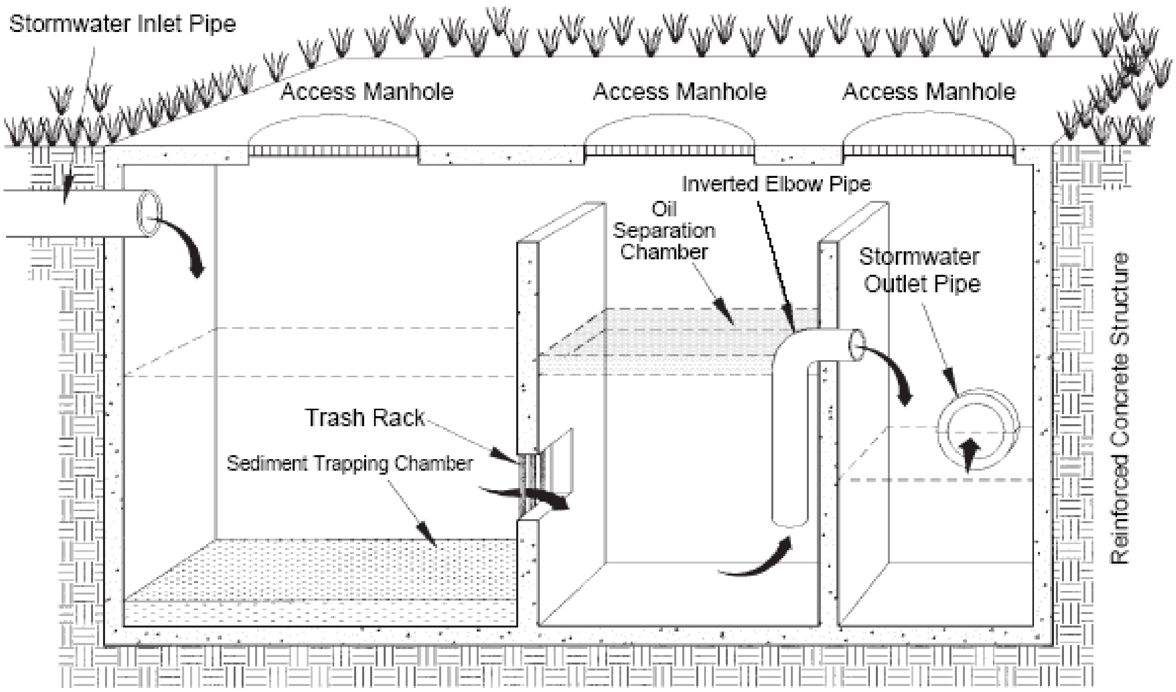
Figure 11-16. Schematic of Typical Oil/Grit Separator



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11.4.4.10 Grate Inlet Inserts

Grate inlet inserts are a type of oil/grit separator consisting of an insert that fits inside a standard grate inlet. Normally the inserts are made of a stainless steel, aluminum, or cast-iron framework which sits on the lip of the inlet grate frame and hangs down into the catch basin inlet chamber. One or more trays of filtration media are placed into the framework. The top screen or tray is usually a sediment trap. The flow enters the top of the filtration tray and filters through.

Filtering media can be made of activated charcoal (for pesticides, fertilizer and metals removal), reconstituted wood fiber (primarily for oil and grease) or household fiberglass insulation. Excess flow beyond the capacity of the media bed or due to media clogging is routed over the sides of the tray(s) and out through the bottom or side of the framework. The capacity of the overflow is designed to equal or exceed the capacity of the grate.

One or more trays of filtering media, sometimes of different types, are then placed either stacked or in a rack below the sediment trap and screen. The media can be disposed in a manner similar to oil and grit chamber sediment though it should be tested periodically to see if it is a hazardous waste.

Typical Applications

a. These can be used in most places where catch basins are installed.

b. It appears to be an ideal application for retrofitting such areas as parking lots, gas stations, vehicle maintenance areas, "dirty" neighborhoods or industrial areas.

Design Considerations

a. Several companies produce such inserts, or they can be fabricated from common materials. The materials which make up the framework and the trays shall be highly resistant to corrosion, easy to install manually, and fit standard inlets.

Advantages

a. It is easy to install, relatively inexpensive, requires no construction or modifications of existing catch basins, relatively easy to maintain by property owners, and is targeted toward the major pollutants from these areas.

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Disadvantages

a. The trapped sediments are potentially toxic.

Maintenance/Monitoring/Enforcement Considerations

a. Proper maintenance should provide a reasonable lifetime vs. costs. There is some question concerning the chemical integrity and longevity of the fiberglass in harsher environments.

b. Maintenance requirements include inspecting the flow integrity of the system and replacement of the filtration media. Quarterly replacement is a good starting estimate though the installations should be checked periodically and after wet periods.

c. Routine maintenance is required and must be built into the cost estimate for the system.

11.4.4.11 Litter Traps

Many designs of litter traps are currently available. Although other litter traps may also be applicable for use in the City, the following seven devices exhibit desirable characteristics.

- Side entry Catch Basin
- The North Sydney Litter Control Device
- The In-line Litter Separator
- The Continuous Deflective Separation Device
- The Baramy® Gross Pollutant Trap
- The Storm Water Cleaning Systems Structure
- The Urban Water Environmental Management Concept

Side-entry Catch Basin Trap (SECT)

The side-entry catch basin has a wire mesh or plastic perforated tray mounted on metal supports embedded in the catch basin side walls; the tray is located immediately inside of a curb inlet, or underneath a grate inlet (see Figure 11-17). Storm water flows through the perforations in the tray, leaving the debris behind. If the perforations are blocked or the tray is full, the storm water flows over the back wall of the tray into the catchpit, and then through the outlet pipe. Removal of the litter is accomplished by either manually cleaning the basket, or the tray is vacuum cleaned and washed with water under high pressure.

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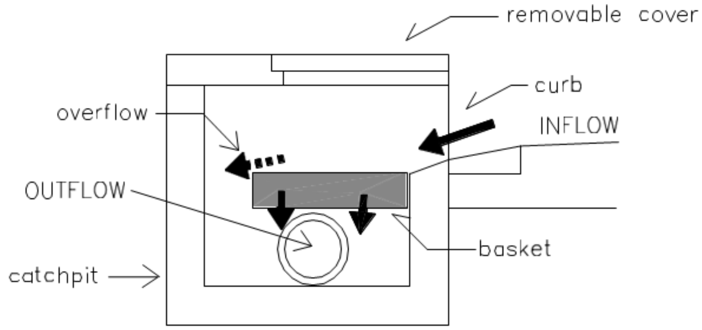
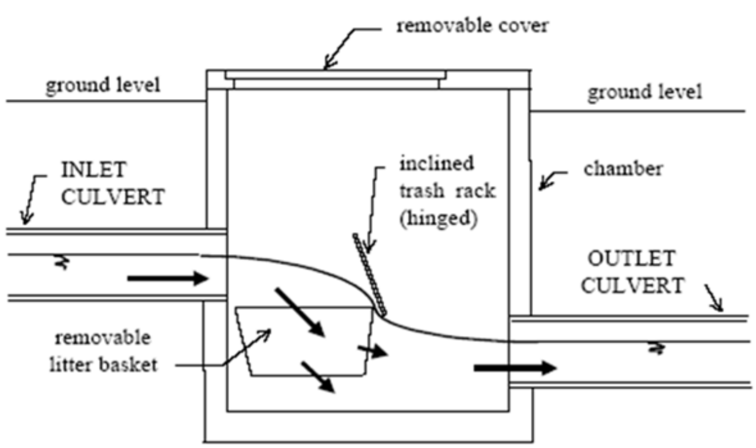
Figure 11-17. Cross-Section through a Typical Side-Entry Catch Basin Trap



The North Sydney Litter Control Device (LCD)

The trap consists of a pre-cast or in-situ concrete pit located downstream of a storm water drainage pipe or culvert. Approximately a three foot drop is provided between the invert of the inflow pipe and the invert of the outlet. This drop provides enough room for removable baskets and a small gap below them. Above the removable litter baskets is a trash rack with vertical bars, which is inclined towards the litter baskets to prevent the inflow from scouring out previously deposited litter (see Figure 11-18). The trash rack is hinged so that it can be pushed back to enable easy removal of the litter baskets.

Figure 11-18. Section through a Typical North Sydney Litter Control Device



The In-line Litter Separator (ILLS)

The In-line Litter Separator is designed to remove litter from underground storm water conduits (up to a diameter of about 30 inches) with minimal head loss. Storm water inflow is diverted from a separator pit into a variable sized holding pit by a carefully shaped boom. Once in the holding pit, the flow is forced down under a suspended baffle wall and up over a weir before being returned to the separator pit downstream of the boom (see Figure 11-19). The relatively large plan area of the holding pit ensures that the average vertical flow velocities are low enough to prevent carry-through of objects, such as plastic bags, that have a negligible settling (or rising) velocity.

In the event of particularly high flows through the storm water conduit, the increased water levels on both sides of the boom cause it to float out of the way. This ensures that upstream flood levels are not affected by the structure, and the litter already trapped in the holding pit is not washed out.

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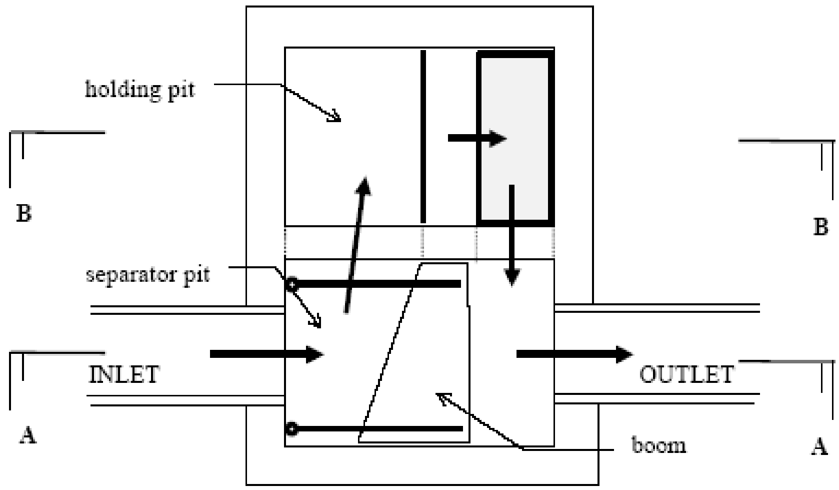
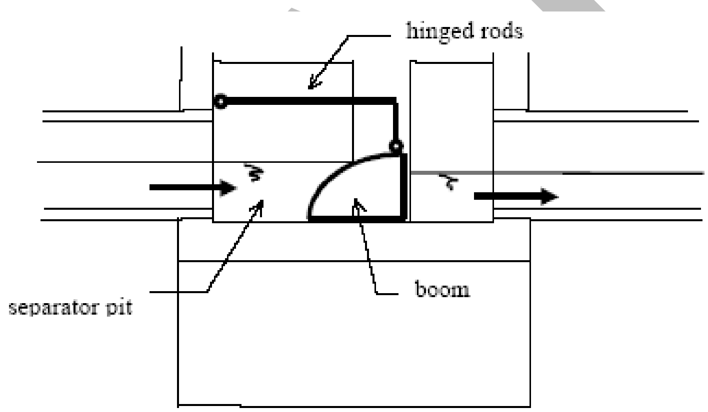
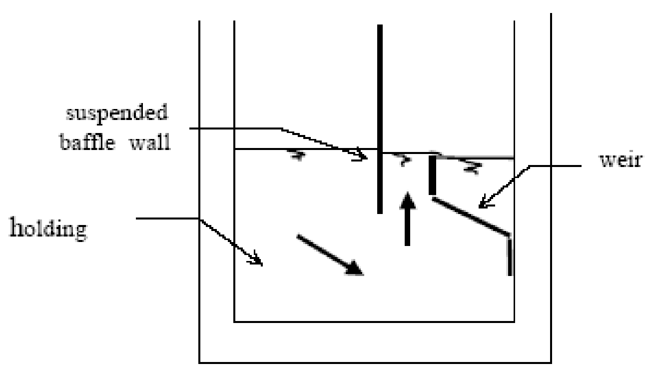
Figure 11-19. In-Line Litter Separator



Plan View



Section A-A



Section B-B

The Continuous Deflective Separation (CDS) Device

The CDS device is an on-line unit that separates and retains both sediments and litter by deflecting the flow and associated pollutants away from the main flow stream of the pipe into a pollutant separation and containment chamber. A diversion weir deflects flow from a storm water conduit into a circular chamber. Gross pollutants are separated within the upper portion of the inner chamber with the aid of a perforated plate screen, which allows the filtered water to pass into a volute return chamber and back to the outlet conduit (see the Horizontal Section of Figure 11-20). The vortex action generated by the incoming flow keeps the water and associated debris contained within the inner chamber in continuous motion. Thus, the litter in the containment chamber is prevented from blocking the perforated plate screen. The heavier pollutants ultimately settle into the lower solids collection sump, while the flotsam floats on the surface of the containment chamber (see the Vertical Section A-A of Figure 11-20).

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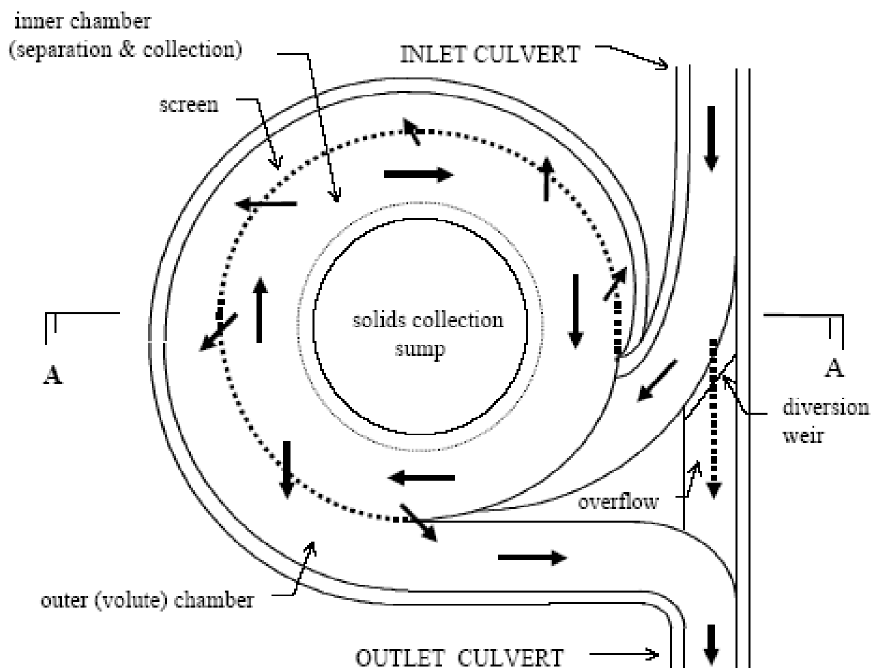
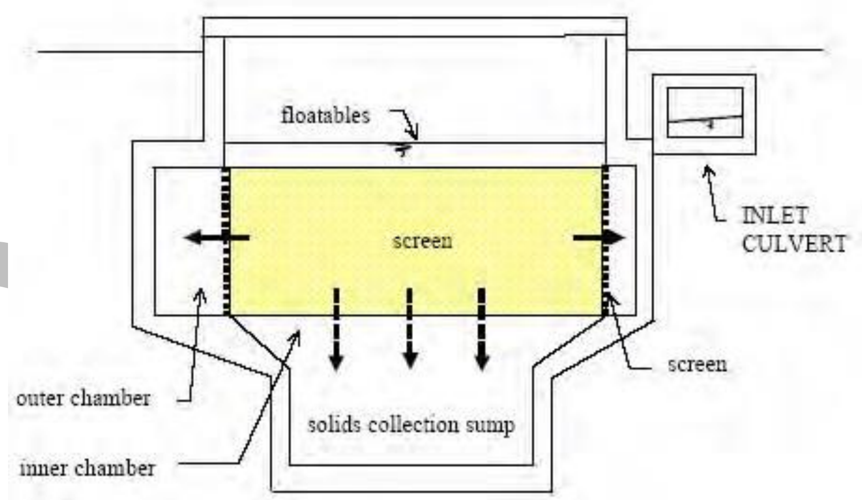
Figure 11-20. Horizontal and Vertical Sections through the Continuous Deflective Separation Device



Horizontal Section



Vertical Section A-A

The Baramy® Gross Pollutant Trap (BGPT)

Baramy Engineering Pty Ltd of Katoomba, New South Wales, Australia developed the simple litter removal structure which has become known as the Baramy® Gross Pollutant Trap. Flow from a conduit is directed over a screen declined at an angle of about 20°; the screen separates the debris from the water and deposits the litter on a collection shelf. The water flows through the screen and either goes under the collection shelf (direct flow version, see Figure 11-21), or is diverted to either side of the shelf (low profile version, see Figure 11-22). After a storm event, the trash can be removed by a skid-steer loader (Bobcat or similar), which gains access down a concrete ramp.

Figure 11-21. Baramy Gross Pollutant Trap – Typical Direct Flow Version

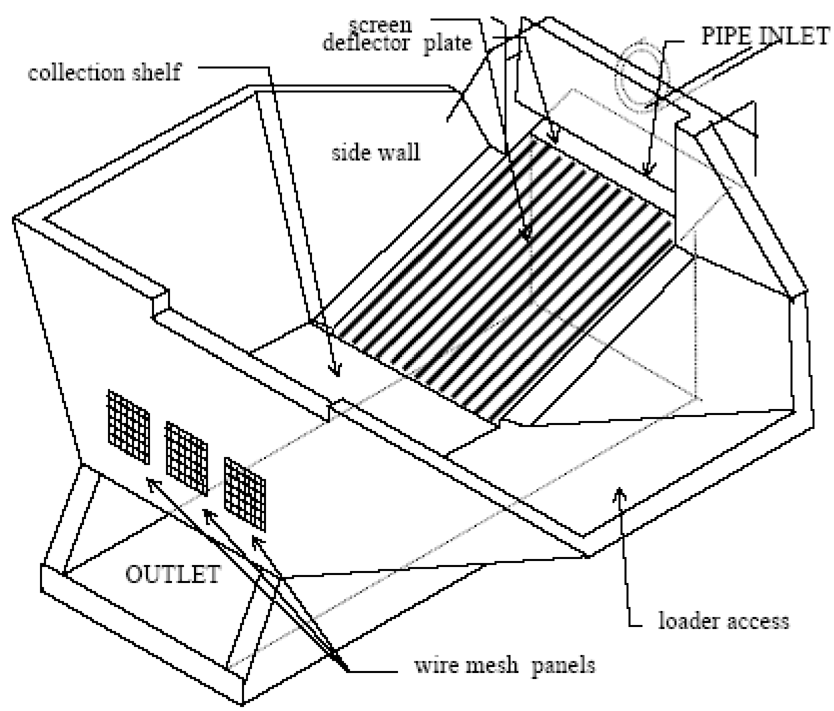
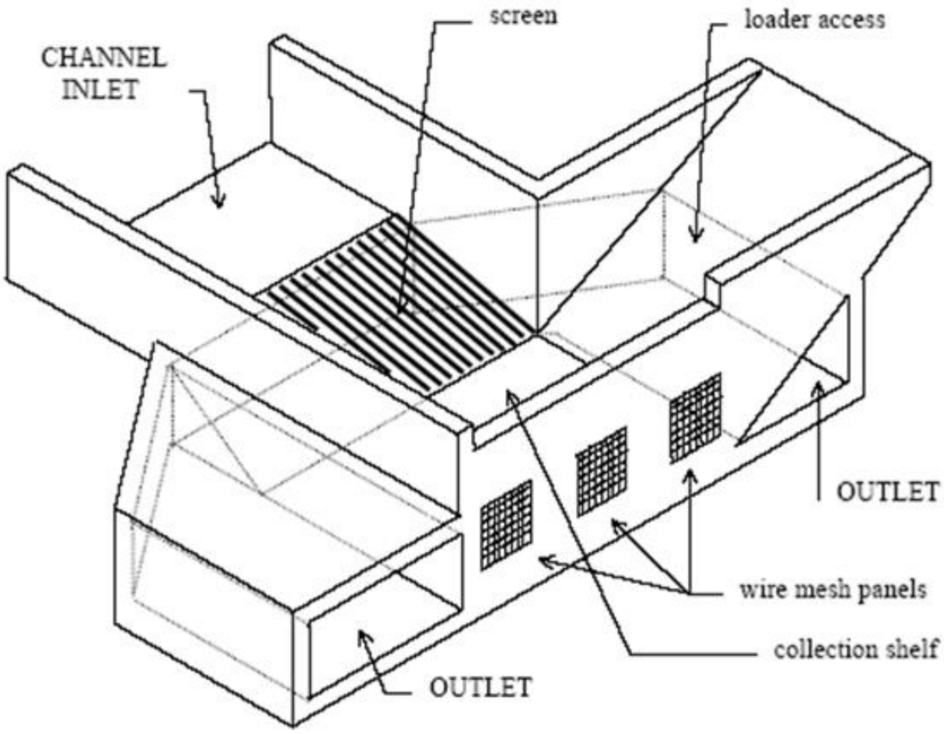


Figure 11-22. Baramy Gross Pollutant Trap – Typical Low-Profile Version



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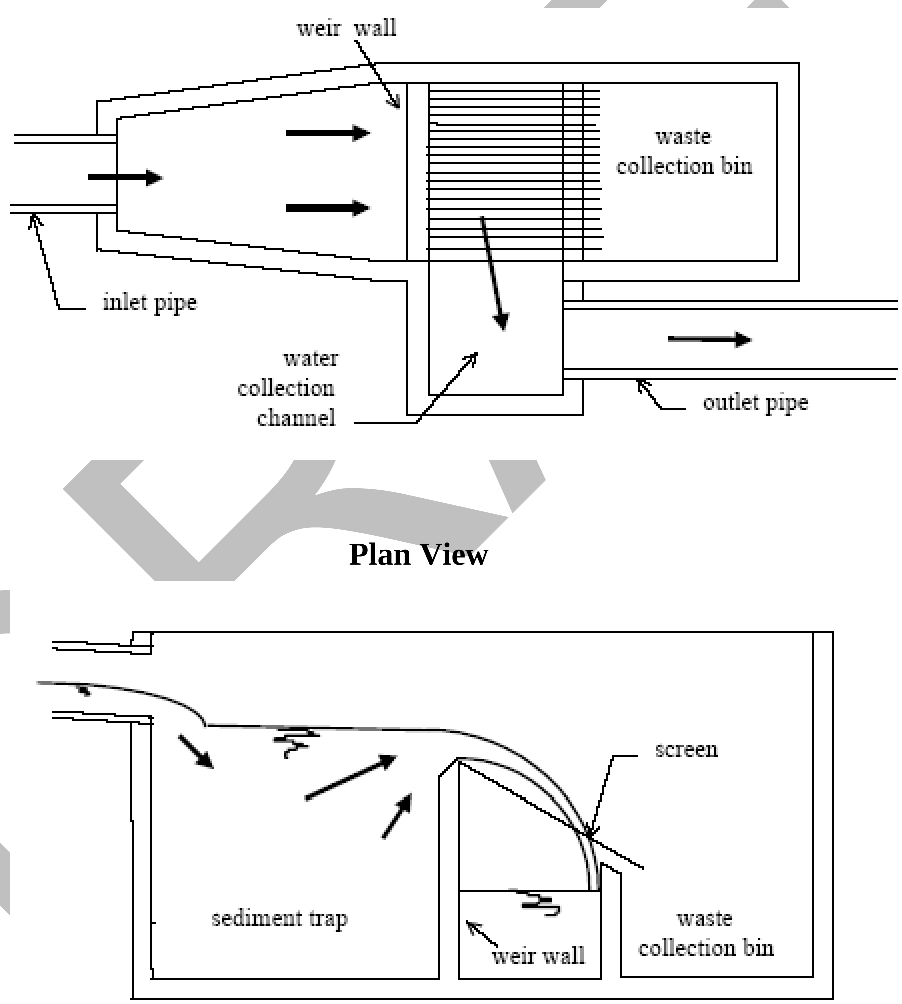
<u>The Storm Water Cleaning Systems (SCS) Structure</u>

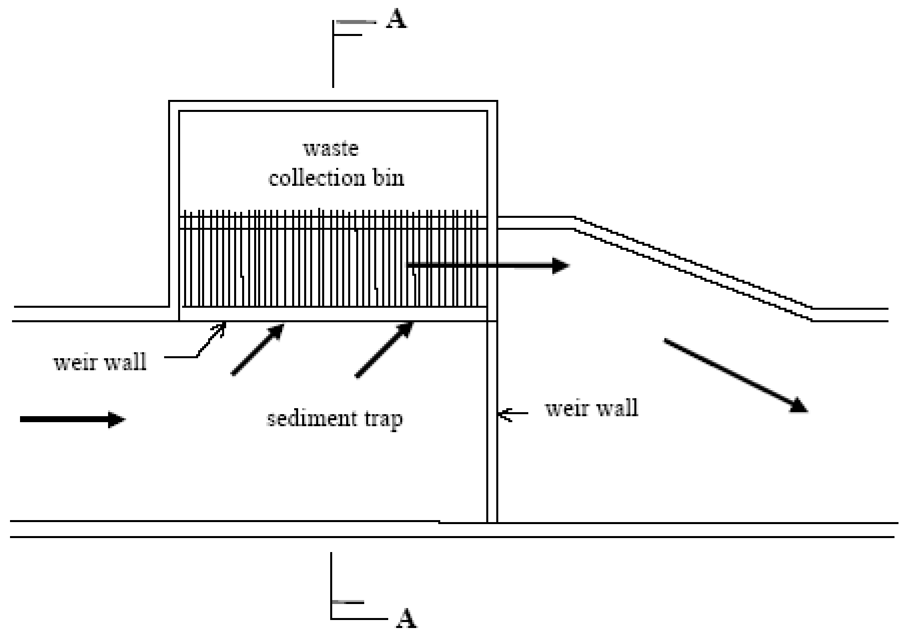
With the SCS structure the water flow is forced over a low weir and through a screen that is declined at approximately 45° below the horizontal. A combination of gravitational forces and the momentum of the water force the litter down the screen, and the trash settles in a bin until removal. There are two alternative layouts. For small flows (from a pipe, for example) the weir is placed directly in the path of flow (Figure 11-23); for flows in canals, the weir is positioned parallel to the initial flow direction (Figure 11-24). If required, a settling basin can be provided upstream of the weir to trap sediments.

Figure 11-23. Storm Water Cleaning Systems Structure for Removing Litter from Pipes

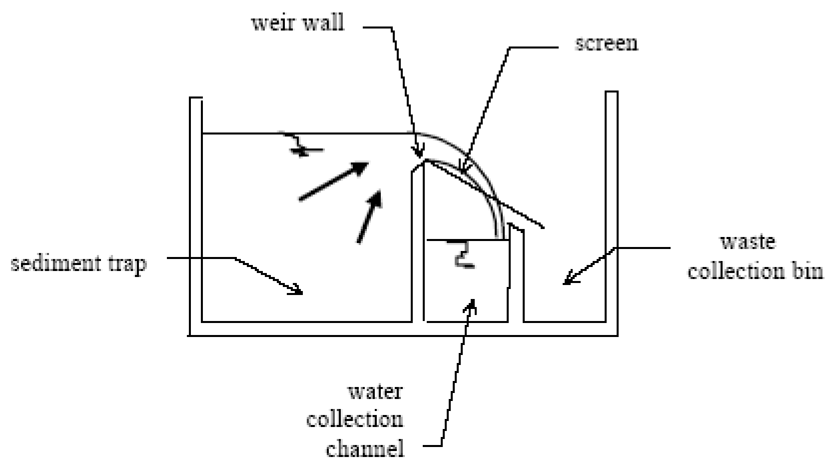


Profile View

Figure 11-24. Storm Water Cleaning Systems Structure for Removing Litter from Canals



Plan View



Profile View

<u>The Urban Water Environmental Management (UWEM) Concept</u>

The Urban Water Environmental Management Concept consists of a hydraulically controlled sluice gate that is used to create the head required to force the storm water through a series of screens, under a suspended baffle wall, and over a weir (Figure 11-25). During a major storm event, the sluice gate automatically lifts to pass the peak and prevent upstream flood levels from rising higher than they would had there been no structure at all.

The device can easily be adapted to remove pollutants other than litter, such as silt, and it can be designed to handle very large flows. Its primary advantage, however, is that it can be applied in areas with flat gradients, such as along the coast, since the head that is required to operate the trap is generated by the hydraulically operated sluice gate.

The trapped litter shall be removed after every storm event. The screens can be cleaned by raising the frame, and the debris either falls or is raked onto the floor of the basin. The litter can then be collected into bags for disposal. Sediment removal will be required less frequently (approximately every three months during the wet season), and it can be cleaned out with a front end loader.

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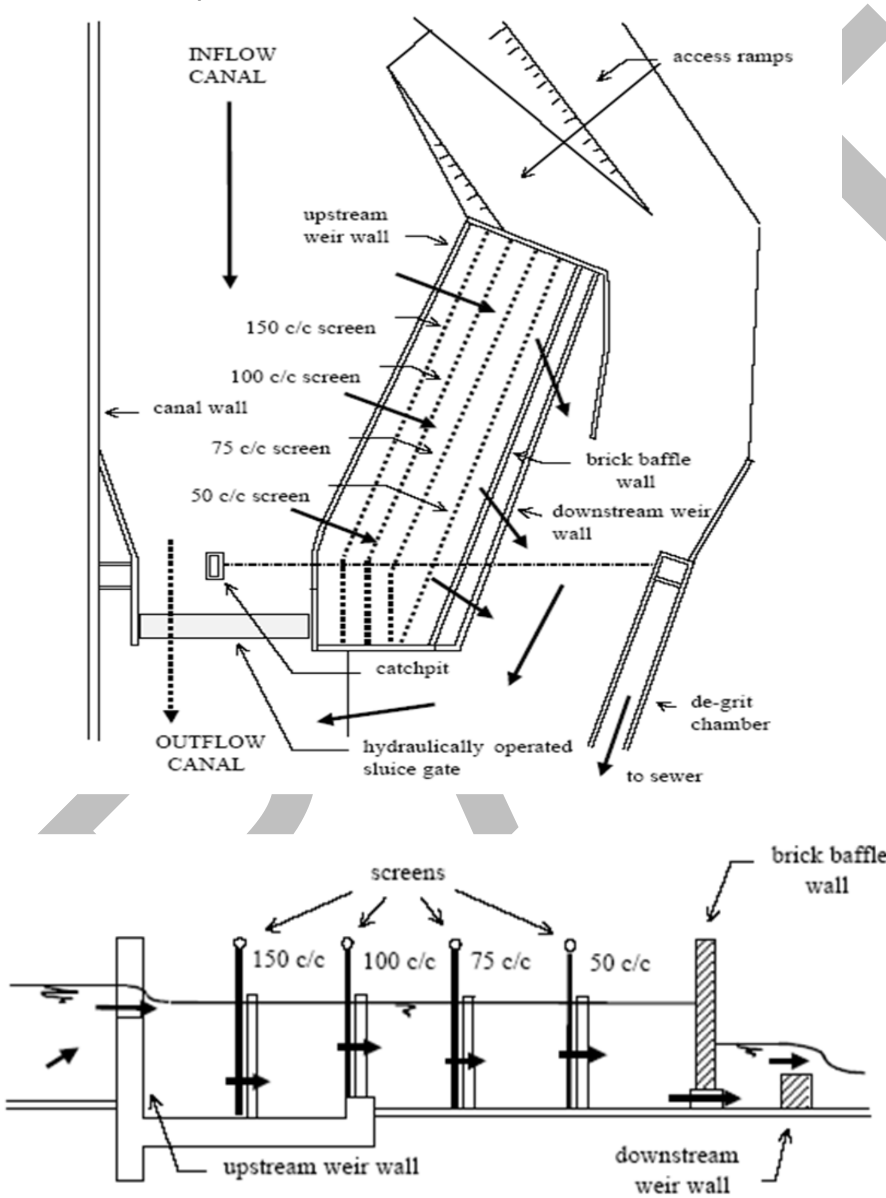
Figure 11-25. Example Plan View and Section of Screens at Canal Control Works



11.4.4.12 Other Manufactured Best Management Practices

The engineer may use other manufactured, structural BMPs, as long as the design parameters in this chapter are met. Maintenance and operation of such BMPs should be taken into account and coordinated with the City.

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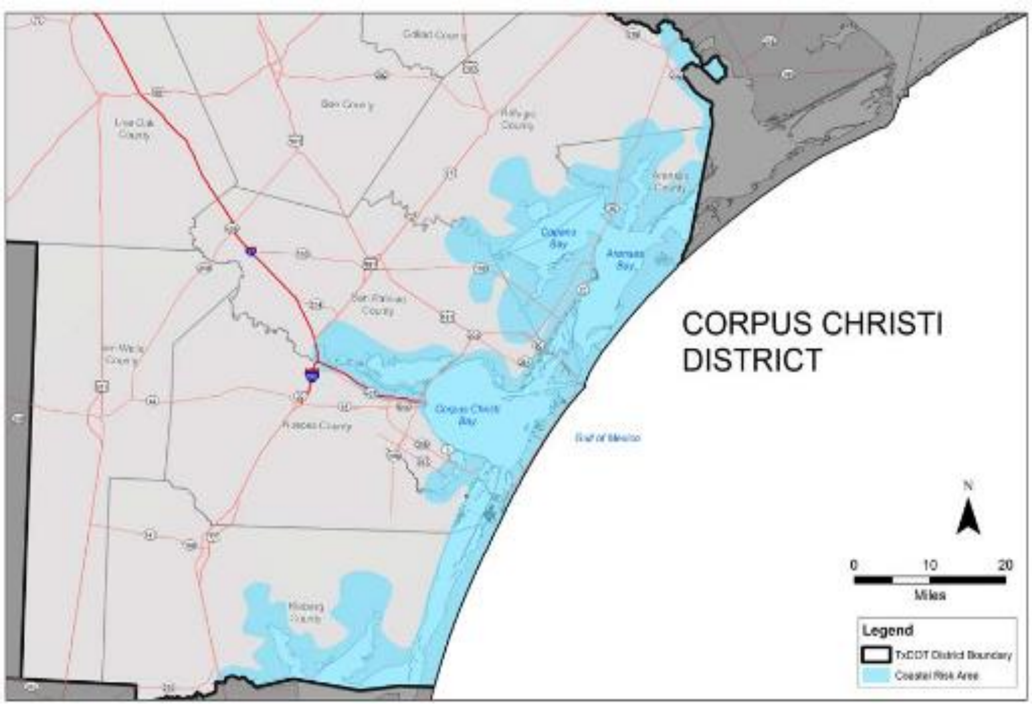
Chapter 12
COASTAL FLOODING

The purpose of this chapter is to provide a general overview of coastal processes, flooding, and engineering considerations including hazards, design considerations, and information regarding construction in a coastal environment. Projects located within the coastal risk area as defined in this chapter should incorporate special considerations throughout the planning, design, and construction processes. For specific design requirements related to constructing and filling in the coastal floodplain, refer to the Flood Hazard Prevention Code (Code of City Ordinances, Chapter 14, Article V).

12.1 INTRODUCTION

The Texas coast includes over 3,300 miles of shorelines and is made up of various ecosystems such as saltmarsh wetlands, dunes, beaches, estuaries, and others. Due to the length of the coastline, variety of ecosystems, and the unique risks for each area, the Texas coast is often split into regions and areas based on each area's characteristics. The Texas Coastal Resiliency Master Plan derived risk areas based on FEMA-designated AE and VE zones and has split the Texas Coast into 4 regions. Corpus Christi falls into Region 3. Note that these risk areas are not set in stone; and if a project is adjacent to an official risk area but would cause similar impacts if the project were inside the zone, the engineer should consider the official risk area expanded.

Figure 12-1. TxDOT Corpus Christi District Risk Areas



The following sections outline considerations for applying design and analysis to projects in coastal areas.

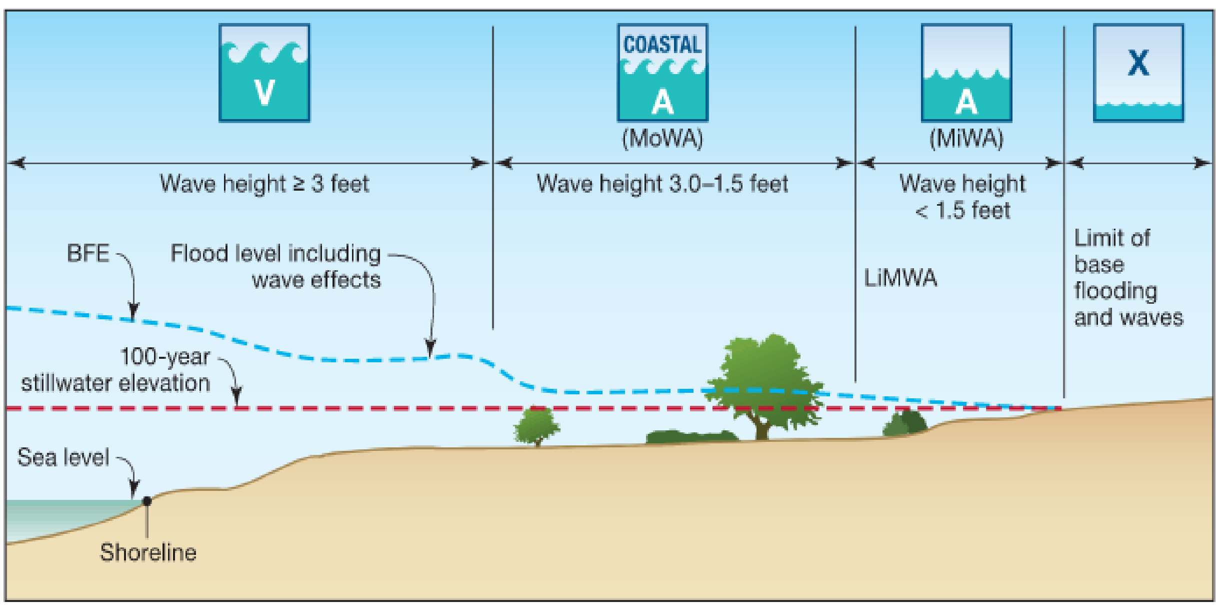
12.2 COASTAL ZONES OF SPECIAL CONSIDERATIONS

Determine if the project is within the TxDOT Corpus Christi District Risk Area, FEMA Coastal A/AE Zone, or FEMA V/VE Zone. If any part of the project is within one or more of these areas, special considerations and elements should be incorporated within the project planning, design, and construction to prevent damaging or impacting the coastal environment and to mitigate the effects of coastal flooding and other damaging naturally-occurring coastal processes from affecting the project.

12.3 FEMA AND THE NFIP

To determine BFEs for areas affected by coastal flooding, FEMA computes 100-year stillwater elevations and wave setup; and then determines the maximum 100-year wave heights and in some areas the maximum 100-year wave runup, associated with those stillwater elevations.

Figure 12-2. Coastal Transect with Stillwater and Wave Crest Elevations



Typical shoreline-perpendicular transect showing Stillwater and wave crest elevations and associated flood zones

12.4 LEVEL OF ANALYSIS DETERMINATION

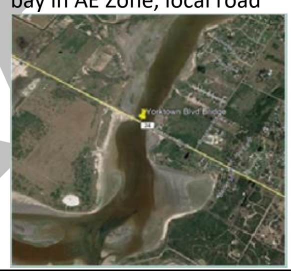
Different projects require different levels of analysis. These levels can be designated as Level 1, Level 2, and Level 3. Some sections will apply to all or some of these levels; this will be mentioned within any applicable section.

Each project will be assigned a risk level designation based on the level of vulnerability and complexity inherent to the proposed project. Risk level designations are the same as those used by TxDOT. A description of each level is taken from Section 15 (15-15 & 15-16) of the TxDOT Hydraulic Design Manual (latest version). Coordinate level of analysis with the City.

DRAFT

Table 12-1 (from the TxDOT HDM): Level of Analysis Required for Representative Coastal Infrastructure Projects

Table 12-1. Level of Analysis for Coastal Infrastructure Projects (TxDOT HDM)

	Level 1	Level 2	Level 3
Approximate Frequency	Most common	Less common	Infrequent
Road Type	Off-system and minor arterials in less critical areas	Various roadway types up to and including interstates in less/moderately critical areas	Highly vulnerable routes, freeways/interstates in very critical areas
Bridge Type	Less critical bridges over a tidal creek in shallow estuary with minimal coastal scour risk	Less/moderately critical bridge that is well-protected with minimal coastal impacts; moderate coastal scour risk	Highly critical or major evacuation route bridges; severe or dynamic coastal scour risk
Vulnerability	Low to Moderate	Moderate to High	High
Road Type Example	Local or minor arterial, less critical roadway 	Principal arterial, but less critical road with armoring needs, located in coastal AE/VE zones 	Interstate with seawall located along major evacuation route 
Bridge Type Example	Culvert or small bridge in AE Zone, well-protected local road 	Small bridge over protected bay in AE Zone, local road 	Causeway connecting mainland and barrier island 
Other Considerations	Less complex analysis may require more conservative design assumptions, which can increase overall cost	More complex analysis may be more time intensive and costly during the design phase, but can reduce overall construction cost for level of protection needed	Complex geometries can cause waves to change direction or height; accounting for these complexities may not be feasible with simpler analytical methods

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12.5 HISTORICAL STORM EVENTS

Regardless of the analysis level, the local extreme storm history should be evaluated. For Level 1 analyses, it's reasonable to compare local high-water marks or similar evidence to published return intervals. The published return interval data should be recent enough to consider recent large storms (e.g.: Harvey).

For Level 2 and 3 projects, a more detailed analysis is required. The following data sources are available to assist in this regard:

FEMA Flood Insurance Rate Maps (FIRM) – Newer FIRMs may include Limits of Moderate Wave Action (LMWA), which represent the approximate limit of where waves reach land at a height of 1.5'. FIRMs and associated Flood Insurance Studies (FIS) can also be used to derive Stillwater elevations.

NOAA Historical Hurricane Tracks – This NOAA database of tropical storm tracks can be filtered based on timeframe, category, radius of interest, etc. Users can find data regarding hurricanes, tropical storms, tropical depressions, and extra-tropical storms. coast.noaa.gov/hurricanes.

NOAA Storm Events Database – This NOAA database contains searchable data regarding tropical and extra-tropical storms. ncdc.noaa.gov/stormevents.

Hurricane Reports – Many state and federal institutions generate reports documenting the impacts and strength of major hurricanes that have impacted the Texas Coast. These include NOAA and USACE on a federal level, and UT Bureau of Economic Geology and the Texas A&M-CC Harte Research Institute on a state level.

Sea, Lake, and Overland, Surges from Hurricanes (SLOSH) Model – This NOAA database contains the results of the National Weather Service's SLOSH model. This model provides estimates of storm surge heights for both real historical storms and hypothetical storms. Storm track, approach angle, speed, tide circumstances, and rotational energy can be varied to assess coastal dynamics. Note that these models do not provide exceedance probabilities for their outputs, rather they express storm surge elevations in terms of storm intensity. The SLOSH database can be seen at nhc.noaa.gov/surge/.

Results from the SLOSH model are meant to be used to evaluate worst case storm surges. While SLOSH results should be evaluated regardless of analysis level, the results are better used for the initial planning phase of a project rather than for design elevations. If the SLOSH model does indicate that a project is vulnerable, consider adding freeboard or armoring to the project.

12.6 NUMERICAL MODELING

Numerical modeling is often not required for Level 1 or 2 but is for Level 3 analysis. This does not mean that numerical models cannot be used in lower-level analyses, but that using them is often more complex than necessary. However, if no good data sources exist for a level 1 or 2 project,

DRAFT

numerical modeling may be required. Numerical models are useful as they can increase the confidence in selected stillwater elevations.

TxDOT outlines commonly used sources for storm tide data relevant to each level of analysis. Some of these sources are numerical models and some are datasets from government organizations. The following table is reproduced from Section 15, Page 24 of the TxDOT HDM.

Table 12-2. Applicability of Storm Tide Numerical Models Based on Level of Analysis

	Level 1	Level 2	Level 3
Design/Modeling Inputs	FEMA flood map elevations, NOAA tide station data, USACE sea level maps, CHAMP outputs	FEMA flood map elevations, NOAA tide station data, USACE sea level maps, CHAMP outputs	2D and 3D hydrodynamic models including coupled wave, storm surge, and morphologic inputs (ADCIRC, Delft3D, MIKE21)

12.7 TIDES AND WATER LEVELS

12.7.1 Stillwater Levels

Stillwater elevations represent the water surface without considering wave runup. Wave runup is defined as the maximum onshore elevation reached by waves, relative to the shoreline position in the absence of waves (from cresting waves, from momentum allowing water to ingress farther on shore, etc.). Stillwater elevations are affected by tidal forces, by storm events, and by longer-term sea level rise caused by global climate effects. Identifying the local stillwater elevation is one of the key first tasks in the analysis of a coastal project.

12.7.2 Astronomical Tides

Tides are defined as the fluctuation of water elevations with the movement of the moon and the sun. This means that generally, the Gulf Coast experiences one high and one low tide per day; these are diurnal tides. Less frequently, there are days that experience two high tides and two low tides per day; these are called semi-diurnal tides. The intra-tidal rise is generally less than two (2) feet along the Texas coast.

Both federal and state resources exist for establishing intra-tidal rise (both accessible via NOAA's dashboard). These can be found at NOAA's Tides and Currents dashboard.
Note that intra-tidal range can differ from place to place along the coast. This can be caused by local wind or current patterns, coastal orientation, and local bathymetry/topography. Due to local effects noted above, some gauge areas may experience amplification and some diminution of intra-tidal ranges.

12.7.3 Spring and Neap Tides

Tide ranges can change throughout the month as the alignment of the sun and moon shifts throughout the month. When the sun and moon are on opposing sides of the earth, their gravitational effects counter each other resulting in lower intra-tidal rise.

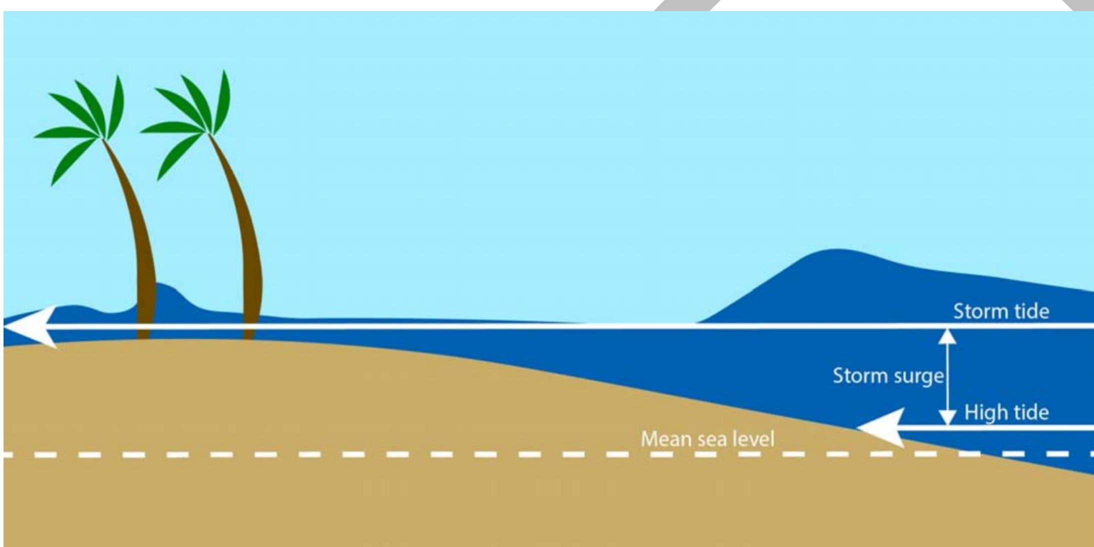
DRAFT

Conversely, when the sun and moon are on the same side of the earth, the gravitational effects can stack. This results in a larger than normal intra-tidal range. It is important to be aware of the annual high tide elevations as it provides an indication of expected water surface elevations during various events.

12.7.4 Tides Amplified by Storm Events

Tropical and extratropical storms can have an additive effect on the stillwater elevation. If the high tide value is representative of a water surface elevation during a storm event, that tide value is known as a storm tide. If there is confidence in the high tide value, an estimate of storm surge height can be calculated. See Figure 12-3 below.

Figure 12-3. Storm Surge vs Storm Tide



Much like rainfall events, storm tides events are also classified based on their return period or exceedance probability. In the storm tide context, this means that a 100-year coastal flood elevation should be exceeded in only 1% of the storm tide events.

There are two main sources of storm tide data that do not entail numerical modeling. The data can be used for both level 1 and level 2 analyses. They can be used for limited purposes for level 3 analysis, but only during initial project evaluation.

a. US Army Corp of Engineers (USACE) – The Galveston USACE District is responsible for engineering projects involving public navigation. Stillwater elevations were developed as a part of these efforts. It is recommended that the District be contacted before using this data so that District personnel can evaluate their applicability prior to use in other projects. www.swg.usace.army.mil/

b. NOAA Extreme Water Levels – Statistically determined storm tide estimates are published by NOAA using data from their network of coastal gauge stations. Within the State of Texas, return periods of 1-year, 2-year, 10-year, and 100-year storm tide

DRAFT

elevations. The 100-year storm tide elevations are often consistent with the 100-year stillwater levels developed as the FEMA base flood elevation.

12.8 VERTICAL DATUM

A vertical datum is a reference elevation above or below which other elevations are measured. Before comparing elevations from different data sources, they must be converted to a common datum.

The North American Vertical Datum of 1988 (NAVD88) is what is often used for elevations on land. Older data is commonly base on the National Vertical Geodetic Datum of 1929 (NVGD29). These datums are based on models of surfaces of equal gravitational strengths and are frequently referred to as orthometric datums. FEMA also has Elevation Reference Marks (ERMs), which are typically found on FIRMs, and can be found here: https://emilms.fema.gov/is_0273/groups/87.html.

Gravitationally defined datums are not the most applicable for tidal datum as coastal elevation statistics will differ spatially. Commonly used tidal datums used in Texas are as follows:

a. Mean High Water (MHW) – Mean of all high-water elevations
b. Mean Sea Level (MSL) – Mean of hourly water levels
c. Mean Low Water (MLW) – Mean of all low water elevations

NOAA has developed a tool that will help convert between these datums called vDatum that can be found here: vdatum.noaa.gov. For any conversions between tidal and orthometric, the conversion will be based on a nearby tidal gauge. Consult with a Registered Professional Land Surveyor (RPLS) or Licensed State Land Surveyor (LSLS) for datum determinations, adjustments, and conversions.

12.9 RELATIVE SEA LEVEL RISE

Relative sea level rise (SLR) and terrain subsidence may be taken into account when designing or analyzing levels of service for long term infrastructure.

Long-term coastal and environmental processes may need to be included in the project's design considerations as these processes can affect local water surface elevations over time. Before determining how to adjust design elevations due to sea level rise, three criteria generally should be met:

a. Is the design life greater than 20 years?

b. Does the project represent a large investment of public funds?

c. Does the project have a low risk tolerance?

Refer to Table 12-3 below (adapted from Table 15-5 in TxDOT's HDM) for screening criteria that should be considered when deciding to incorporate sea level rise into a project's design.

DRAFT

Table 12-3. Screening Criteria for Incorporating Sea Level Rise

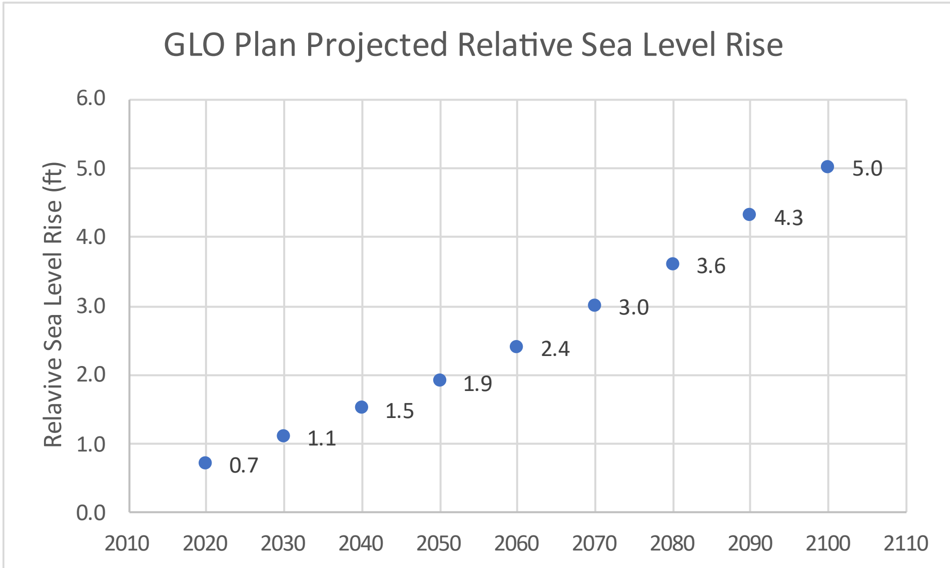
Factor to Consider re Incorporating SLR	Critical	Non-Critical
Project Design Life	Long (>20 years)	Short (<20 years)
Redundancy/Alternate Routes	No redundant/alternative route	Existing redundant/ alternative route
Anticipated Travel Delays due to SLR	Substantial delays	Minor or no delay
Criticality to Movement of Goods	Critical	Not critical
Criticality to Evacuation/Emergency Services	Critical	Not critical
Criticality to Traveler Safety	Critical	Not critical
Expenditure of Public Funds	Large investment	Small investment
Interference with Adjacent Projects/ Infrastructure	• Streets and roadways will require substantial modification • Environmentally sensitive areas/ projects likely impacted • Anticipated SLR would impact infrastructure, property, or other water bodies	• Adjacent streets and roadways will be unaffected • Environmentally sensitive areas/ projects not in vicinity • Anticipated SLR would not impact infrastructure, property, or other water bodies

In 2023, the Texas General Land Office (GLO) updated their Texas Coastal Resiliency Master Plan (GLO Plan). Among other things, the sea level rise predictions were updated to reflect projections released in the 2017 NOAA Global and Regional Sea Level Rise Scenarios for the United States report which was in turn informed by the fifth report from the International Panel on Climate Change.

Where NOAA reports an intermediate scenario of a 3.3-foot global mean sea level increase by 2100, the GLO Plan disaggregates this 3.3 value into regional zones within Texas. Corpus Christi is located within GLO Plan Region 3 spanning from Baffin Bay to the south and Copano Bay to the north. The following chart outlines the relative sea level rise projected in the GLO Plan.

DRAFT

Figure 12-4. GLO Projected Relative Sea Level Rise



Notes:

a. Projections are relative to 2000 and should be modified to a baseline for the estimated rise that has occurred between 2000 and the project start year to estimate the rise expected during the project life.

b. Projections are regional averages of projected rises in the GLO Plan and include anticipated local subsidence.

c. Alternate Relative Sea Level Rise scenarios and projections may be considered if it is determined that they are more appropriate for a given project's level of risk.

d. Data from Table 15-6 in TxDOT HDM (2019).

12.10 EXPECTED DESIGN LIFE

When including projected relative sea level rises in a project's design, first determine the expected design life of the project. This can be derived from City requirements, the TxDOT Hydraulic Design Manual, the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications, International Building Code (IBC), or based on the goals and purposes specific to the project in question.

As an example, consider the installation of roadway with a design life of 20 years starting in 2030:

$$RelRise = Rise@2050 - Rise@2030 \quad \textbf{Equation 12-1}$$

$$0.8' = 1.9' - 1.1' \quad \textbf{Equation 12-2}$$

The calculated relative rise of 0.8' can now be added to the design elevation of the pavement.

DRAFT

The following steps should be considered before applying and using relative sea level rise forecasts:

a. Obtain elevations for the project (as-builts, survey, proposed conditions, etc.).

b. Calculate the projected relative sea level rise based on your project's lifetime and the above chart (be sure to modify these numbers to a more relevant baseline than 2000).

c. Add the calculated rise from step 2 to the elevations from step 1 to obtain forecasted water surface elevations.

d. Compare the projected water surface elevation to the regulatory floodplain data (100-year storm surge, Limits of Moderate Wave Action (LMWA), etc.). If FEMA elevations are higher than the calculated rise, the use of projected sea level rise may not be necessary.

e. Identify any negative impacts of the relative sea level rise. Does the scour risk increase with higher forecasted water surfaces? Will drainage systems lose efficiency as the downstream elevation is higher?

f. Assess if adaptive measures are necessary for the given negative impacts. Will the impact be temporary? Could a short-term road closure be enough of a mitigation?

g. Assess what adaptive measures may be necessary for the given negative impacts. Once these adaptive measures are identified, it is important to perform a cost analysis for action and inaction. Ensure that these decisions are documented.

h. Consider unintended hydraulic impacts when designing with relative sea level rise in mind. As an example, raising a roadway profile might result in impeded drainage of runoff which might affect adjacent properties. All City design requirements must be taken into account and be met or exceeded.

Below are several sources regarding relative and absolute sea level rise:

a. Texas General Land Office 2019 Texas Coastal Resiliency Master Plan (coastalstudy.texas.gov/resources/files/2019-coastal-master-plan.pdf)

b. Federal Highway Administration HEC-25, Highways in the Coastal Environment, Volume 3 (https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif19059.pdf)

c. NOAA Global and Regional Sea Level Rise Scenarios for the United States, 2017 (tidesandcurrents.noaa.gov/publications/techrpt83_Global_and_Regional_SLR_Scenarios_fot_the_US_final.pdf)

DRAFT

12.11 DESIGN STILLWATER LEVELS

Stillwater elevations are the basic starting point for the calculation of design elevation. The remaining portion of the design elevation will be discussed in the next section about waves. The stillwater elevation is a combination of the astronomical tides and storm surge. The stillwater may also need to consider relative sea level rise if the project lifetime calls for it. Determination of the return period for the project is necessary to determine what data will be relevant for determining a stillwater elevation for projects where Level 1 or 2 analyses are required, it may be acceptable to combine relative sea level rise to a FEMA storm tide from the relevant FIS (note that it is important to exclude wave heights from data taken from FIS).

More complicated projects requiring Level 3 analysis require additional effort. These models will incorporate many variables such as wind speed, air pressure, landfall geometry, and more into a stochastic model to determine probabilities. These models can also be used to continuously evaluate the project as well as incorporate changes to assumptions.

12.12 WAVES AND CURRENTS

Previously discussed parameters inform long-term and large-scale processes that determine wide-ranging average values, however not all coastal risks are based on these. Additional parameters determine hyper-local processes that cause issues. These are generally known as nearshore processes. The primary causes of these are waves and currents.

12.12.1 Waves

Most waves visible at the shoreline are wind derived. As the wind blows parallel to the water surface, the non-slip condition between the air and water tends to drag the water in the direction of low elevation bulk air flow. This process is constant and only really changes when the patterns of bulk air movement change close to the water surface or when the water encounters a (mostly) immovable object such as a coastline.

During storm events, the high wind velocities can impart additional dragging force, giving more energy to the waves. These storm-enhanced waves can travel and carry energy over vast expanses of ocean. As these waves encounter elevated terrain, they will break earlier, with more energy to impart, and crest higher than usual. As the enhanced waves can introduce water farther inland than waves are usually able to, it may be necessary to consider coastal erosion impact farther landward.

The most basic approximation of wave dynamics is a sinusoidal function. This approximation assumes that there is no interaction with other fluid motion, the wave's path of motion is free of obstructions, among others. In reality, wave behavior is much more complex; it is a result of many processes that act on various scales of both time and geography.

Wave impacts will be different at locations adjacent to open coastline versus locations subject to local heterogeneity such as inlets, bays, shipping channels, and barrier

DRAFT

islands. Long term averages of wave gauge data can help generate wave characteristics at these locations.

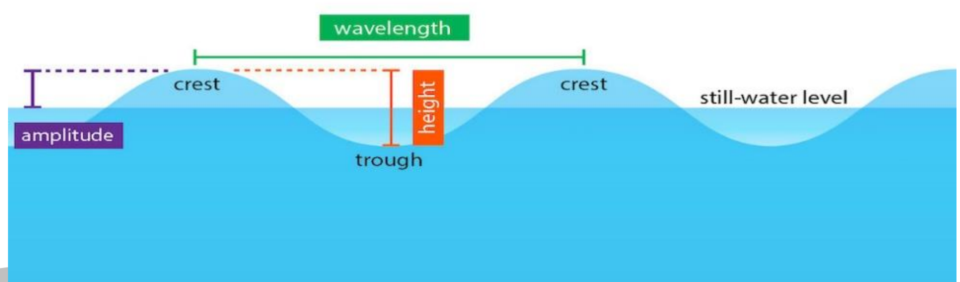
12.12.2 Wave-Generated Currents

Waves in the Gulf of Mexico are generated primarily by wind. At the location of a storm, waves tend to propagate in various directions. As the wind sets up into a current, the waves will begin to sort into a single direction. The distance across open water where the wind generates waves is called the fetch.

After the waves are set up and established, they tend to generally follow a mathematical model. The basic parameters of wave dynamics are:

- Wavelength (L) – the horizontal distance between wave crests
- Wave Height (H) – the vertical distance between wave crest and trough
- Wave Height Upper (Y) – the vertical distance from the stillwater level to the crest height
- Water Depth (d) – the depth of the water at the stillwater elevation
- Wave Period (T) – the time interval between crests of the wave

Figure 12-5. Diagram of Wave Parameters



12.12.3 Obtaining Design Wave Heights from Gauges

If your project involves establishing wave heights by way of a gauge, there are important statistical considerations. It may seem reasonable to design to the maximum wave crest elevation seen over the course of data collection, however it is common to instead use an average of the highest third of all crest heights. This statistical aggregation is an attempt at disregarding outliers that can add significant undue cost to a project budget. The standard wave pattern sampling time for a representative sample of wave actions is 20 minutes.

DRAFT

12.13 GENERAL WAVE PHYSICS

Wave Breaking

When waves approach a nearshore zone, waves undergo a process called shoaling. The shoaling process increases the wave height. As the wave continues landward, the height increases (and stillwater depth decreases) to a point at which the wave collapses and breaks.

If waves have enough energy, the forward momentum of the wave crest will carry the wave break forward. This is what creates the stereotypical tubular wave. Commonly, this occurs when the wave height is approximately equal to twice the stillwater depth.

Waves break in a very turbulent fashion; a breaking wave can impart substantial energy on a structure. Design of armoring or coastal revetments will need to be in part based on an estimate of wave crest. In a Level 1 situation for which numerical modeling is required, the following standard method of estimating wave height can be used:

$$Y_{max} = 0.6 * \mathrm{d}$$ **Equation 12-3**

where: Y_{max} = the maximum wave height possible at d
d = the depth of water including storm surge and tides

Wave Refraction

Waves approaching the coastline at an angle will undergo refraction. As the depth decreases, the waves will move more slowly and the wavelength decreases. The direction of the wave's motion will also become closer to being perpendicular to the section of coast that it will impact. An irregular subsea terrain can lead to complex refraction patterns, leading to much of the irregularity in coastal data between coastal locations.

Wave Diffraction

Diffraction is the process by which an obstruction in the path of a wave will cause the wave to spill around the corners of the obstruction in addition to continuing its previous path. Diffraction is an important process to consider as it can alter the direction of wave propagation.

Wave Reflection

Ocean waves reflect when they encounter wide vertical surfaces. Depending on the energy contained in the wave front the reflection can cause complex wave interactions. These wave processes can alter local wave patterns which are critical considerations in the placement of coastal armoring.

DRAFT

12.14 GULF INTRACOASTAL WATERWAY

The coast of Texas is lined with a chain of barrier islands that separate bays and other coastal landforms from the Gulf of Mexico. This waterway that extends east into Louisiana and beyond allows for shipping and travel between coastal states without having to navigate into the deeper and potentially rougher Gulf waters. These barrier islands provide opportunity for wave reflection, refraction, and diffraction to occur. Padre, Mustang, and San Jose Islands serve as barriers between Corpus Christi and the wider Gulf. helping reduce coastal erosion of the mainland by absorbing energy from waves or storm surge. Development on these barrier islands can hinder their effectiveness as overtopping can cut preferential flow channels through them. Improvements on the island should not reduce the presence or effectiveness of the dune system.

AASHTO's Guide Specifications for Bridges Vulnerable to Coastal Storms, 2023 (AASHTO Guide) recommends adding at least one (1) foot of freeboard above the 100-year design wave crest elevation so that impacts between bridge low chords and breaking waves are minimized.

12.15 DESIGN WAVE CREST ELEVATION

Wave crest elevations can be vital in determining the elevation of a roadway or bridge. HEC-25 from the USACE contains a detailed approach for calculating design wave crest elevations, reproduced below:

a. For a Level 1 analysis, waves can be assumed to be depth limited as is common along the Texas coastline. As such, the design wave height above the stillwater level can be calculated using Equation 12-4 below. This value is added to the stillwater elevation (SWE) to obtain a design wave crest elevation (DWCE). More explicitly, this is:

$$DWCE = SWE + 60\% * \mathrm{d}$$ **Equation 12-4**

b. For a Level 2 or 3 analysis, design wave crest elevations will need to come from numerical models that rely on local bathymetry and topography, local wind patterns, local stillwater elevations as inputs.

If the wave crest elevation cannot be avoided, further analysis is required to either mitigate or accommodate these forces. Sections 4.3 and 4.4 of the AASHTO Guide outline appropriate levels of analysis and design strategies depending on an assessment of criticality/importance.

If a bridge is considered Extremely Critical, it will need to be designed such that wave forces cause minimal to no damage. Service restoration for Extremely Critical bridges needs to be immediate. If a bridge is considered Critical, it will need to be designed to withstand a design storm, but some longer-terms repair work should be expected.

AASHTO defines the two criticality categories as:

a. Extremely Critical – Bridges that are required to be open immediately after the design event for emergency vehicles and quickly open to public use. Note that all formally designated evacuation routes should be included in this definition.

b. Critical – Bridges that should be open to emergency vehicles shortly after a design storm and open to the public within days.
   Details regarding the determination of bridge criticality and analysis level can be found in the AASHTO Guide. Below are a select list of considerations that will be necessary (recreated from FDOT, 2009):

   i. Age and condition of the existing bridge structure and the feasibility/cost of retrofitting to resist wave forces
   ii. Proposed bridge elevation and location alternatives
   iii. Estimated cost of elevating structures above wave crest clearance; why might this be infeasible?
   iv. Construction cost increases due to adjusting elevation or location
   v. Will local routes be impacted?
   vi. Are there detours (redundancy)?
   vii. Evacuation/EMS routes

12.16 LACUSTRINE WAVE GENERATION

In much the same way as waves are initially generated and then filtered into bulk wave motion in a wide-open ocean setting, similar processes can generate waves in lakes given the right conditions. The wave height is a function of wind energy and fetch length. Persistent lacustrine waves can erode shorelines in much the same way as ocean waves can erode coastlines.

Just as barrier islands and engineered dunes can protect coastlines, establishing vegetated zones along the vulnerable shoreline can be beneficial.

12.17 SHIP WAKES

Ship wakes are a common source of waves in sheltered water bodies. Wave crest heights of these waves are a function of the vessel's speed, sizes, shape, and distance from shoreline or coastline. Generally, large ships can generate waves with heights more than 10 feet while less energetic vessels like barges and tugboats generate waves of only 5 feet. Wake waves from the most energy intensive vessel should be considered in design. The following studies present methods for estimating wake waves:

a. Large Vessels – Weggel, J. R. and Sorensen, R. M., 1986. Ship Wake Prediction for Port and Channel Design. Proceedings from the Ports 1986 Conference. American Society of Civil Engineers. P. 794-814

b. Large Vessels – Kriebel, D. L., and Seelig, W., and Judge, C., 2003. A Unified Description of Ship-Generated Waves. Proceedings of the PIANC Passing Vessel Workshop. Portland, Oregon.

c. Small Watercraft – Bottin, R. R. Jr., McCormick, J. W., and Chasten, M. A., 1993. Maryland Guidebook for Marina Owners and Operators of Alternatives Available for the Protection of

DRAFT

Small Craft Against Vessel-Generated Waves. Prepared for the Maryland Department of Natural Resources. Coastal Engineering Research Center. Vicksburg, Mississippi. 92pp.

Numerical modeling should be used to evaluate wave cresting impacts for projects where complex nearshore processes are involved or where Level 2 or 3 analysis is warranted. Regardless of analysis level, the following sources of wave data may be helpful.

NOAA National Data Buoy Center – Oceanographic gauging stations record wave data throughout the Gulf of Mexico (ndbc.noaa.gov).

USACE Wave Information Studies – Long term wind and wave data can help engineers determine statistically based parameters. These datasets span the entirety of the Gulf coast (wis.erdc.dren.mil).

FEMA Flood Insurance Studies – Some FEMA FISs provide estimates of wave height above the coastal stillwater elevation. Values from these studies should not be used for the more detailed Level 2 or 3 analyses, however they are fine for use in Level 1 analyses.

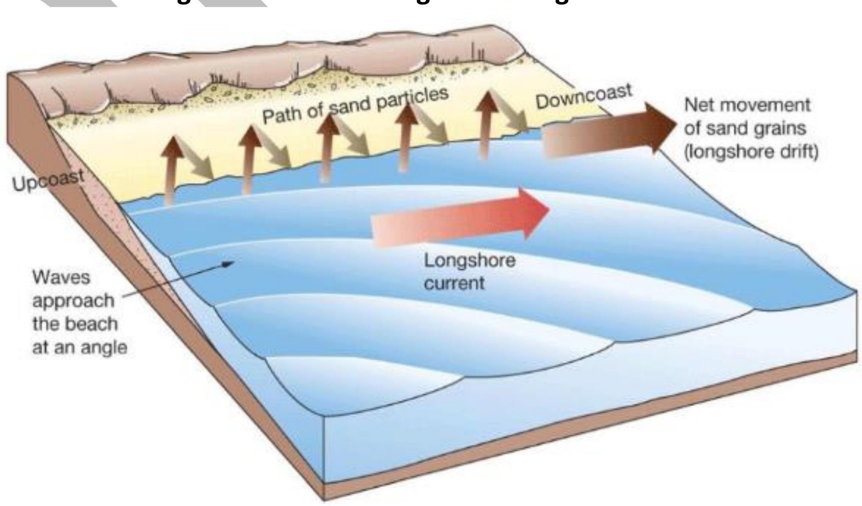
12.18 CURRENTS

Waves can dissipate energy in two ways, the first is through the wave crest breaking as described above. The second is by way of creating currents. Understanding how currents are generated and how they move water adjacent to your project is critical, as they can move sediment around below the water surface. These currents can scour out preferential pathways, altering the local subsurface terrain.

12.18.1 Longshore Currents

Most incoming waves do not impact perfectly parallel to the shore. Because of this angle, there is a periodic resultant force that will, over time, move sand grains from upcoast to downcoast. See the figure below, taken from the TxDOT HDM.

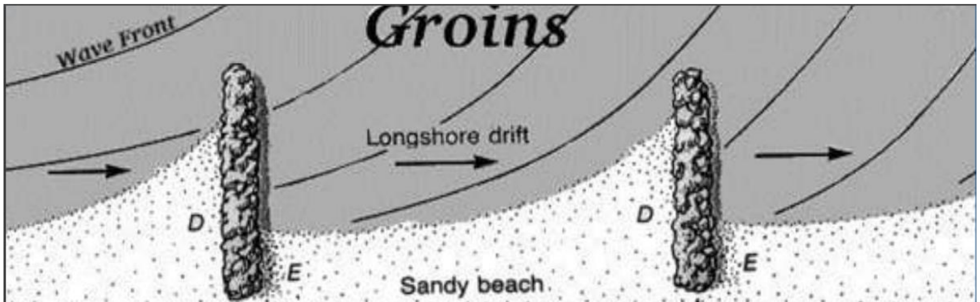
Figure 12-6. Wave Angle and Longshore Current



Storm events can exacerbate this process, leading to beach erosion. This process is best seen when countermeasures have been installed. Anchored protrusions called groins protruding from the coastline block the eroded beach leading to depositional environments upcoast from each groin. This can be seen in the figure below. If infrastructure is placed in the erodible area, the beachy substrate can erode, leading to scour issues.

DRAFT

Figure 12-7. Beach Erosion Control Groins



North of Padre Island in Kenedy County, longshore currents generally flow from north to south. South of Kenedy County, longshore currents generally flow northwards. Consequently, the Kenedy County coastline, known as the Coastal Bend, is considered sediment rich. Texas beaches on the upper or lower stretches of coastline are considered sediment poor.

A common equation for estimating sediment transport rates from longshore currents is known as the energy-flux method (sometimes called the CERC Equation). This equation is best for determining shorter term sediment budgets. For longer term or high energy impact analysis, it is often easier to analyze aerial imagery through time. For details on the CERC Equation or more information, refer to the HEC-25 document.

12.18.2 Rip Currents

As waters recede from the beach, rip- or cross-shore currents can be a source of danger. These can be caused by the normal periodic wave motion, by post-storm surge retreat, or by the collision of two opposing longshore currents. These currents can pull sand from the shoreline, often creating recessed areas that erode faster than the surrounding beach. In turn, this can decrease the buffer distance between infrastructure and incoming waves or storm surge.

Numerical modeling should be used to evaluate rip currents and their impacts for projects where complex nearshore processes are involved or where Level 2 or 3 analysis is warranted.

Both cross- and long-shore currents can cause scour at bridges or roadways. Similarly, infrastructure in the coastal zone can have impacts on current-based aggradation and degradation far downcoast from the project site. To mitigate impacts, engineered approaches such as jetty installation, channel dredging, or vegetated momentum barriers can be used, however their downcoast impacts need to be evaluated. Coordinate any such improvements with the City for submittal and approval requirements.

DRAFT

12.18.3 Setting a Design Elevation

The design elevation is computed by adding the required freeboard to the design wave crest elevation.

$$DE = \mathrm{DWCE} + \mathrm{FB} \qquad \textbf{Equation 12-5}$$

In coastal environments, it may not be practical to design a bridge deck or roadway such that it sits above a design elevation. In these cases, it is vital to evaluate where and when there need to be usage limitations communicated to stakeholders.

Bridges may have other requirements that dictate the height of the bridge. These requirements will likely supersede the typical design elevation calculation.

- Low chord (deck invert) elevations above the typical design elevation might be required at ship channels to provide clearance for ships or elsewhere to clear potential debris that receding storm surges may bring
- Low chord elevations lower than the typical design elevation might be required due to the

For a walkthrough of how to set a design elevation for each level of design complexity, see Tables 12-4, 12-5, and 12-6 below (recreated from tables 15.7, 15.8, and 15.9 in the TxDOT HDM(2019)).

DRAFT

Table 12-4. Level 1 Analysis

Level 1 Analysis Example	
Determine Storm Tide Elevation	**Obtain appropriate FEMA flood map elevations for study area** FEMA FIS Report Summary of Stillwater Elevations of 10 ft for 100-year design storm tide and 8.4 for 50-year design storm tide Zone AE BFE shown on Flood Insurance Rate Map includes wave effects and runup and should not be used for stillwater elevation, reference the FIS for stillwater elevations
Determine Relative Sea Level Rise Rate	**Obtain relative SLR rates for project life cycle** Located in Nueces County, Region 3; assumed 50-year design life for roadway and bridge starting in 2020; relative SLR for 2070 is 3.0 ft per Section 2 guidance
Interpolate Project Specific Relative Sea Level Rise	• the 3.0 ft for 2070 minus 0.7 ft from 2020; SLR of 2.3 ft for project life
Calculate Stillwater Elevation	**Add the result of Step 3 (relative SLR) to the elevations obtained in Step 1 (storm tide)** • 100-year: 10 ft plus 2.3 ft gives a stillwater elevation of 12.3 ft • 50-year: 8.4 ft plus 2.3 ft gives a stillwater elevation of 10.7 ft
Obtain Ground Data	**Obtain appropriate ground surface elevation maps for study area** LiDAR and bathymetry from TNRIS show an average channel elevation of 7 ft; ground elevation should be confirmed with plans, survey, or bathymetry
Determine Stillwater Depth	**Subtract ground elevations (Step 5) from values found in Step 4 to obtain flood depth** • 100-year: 12.3 ft minus 7 ft equals a depth of 5.3 ft • 50-year: 10.7 ft minus 7 ft equals a depth of 3.7 ft
Determine Maximum Wave Height	**Multiply stillwater depths (Step 6) by 0.8 to determine maximum wave height*** • 100-year: 5.3 ft times 0.8 equals 4.24 ft wave height • 50-year: 3.7 ft times 0.8 equals 2.96 ft wave height
Determine Wave Crest Elevation (Design Elevation w/out Freeboard)	**Multiply maximum wave height (Step 7) by 0.75, add to Step 4 for wave crest elevations; 75% of the wave height being above the stillwater elevation is a standard estimate for basic wave calculations (HEC-25)** • 100-year: 4.24 ft times 0.75 plus 12.3 equals 15.48 ft • 50-year: 2.96 ft times 0.75 plus 10.7 equals 12.92 ft Significant discrepancies between these values and historical storm observations; if the discrepancies are large, a Level 2 analysis may be required
Determine Bridge Design Elevation (Low Chord)	**Determine bridge low chord elevation^** Per TxDOT practice, a freeboard of 2 ft is added to the 50-year design wave crest elevation • 12.92 ft plus 2.0 ft equals 14.92 ft
Determine Roadway Design Elevation	**Determine local roadway design elevation^** Per TxDOT practice and AASHTO guidance, local roads do not require freeboard above the 50-year design wave crest elevation

* If the project is located adjacent to a FEMA coastal transect, it is also possible to determine appropriate wave parameters from the Flood Insurance Study documentation. If wave heights significantly exceed the FEMA designated zone, a different method should be considered, such as Level 2 techniques. For example, if landward of the LiMWA, wave heights should likely not exceed approximately 1.5 feet, and if in an AE zone rather than a VE, they should not exceed 3 feet. Inclusion of relative SLR could impact FEMA data applicability in some cases.

^ Application of wave effects, relative SLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.

DRAFT

Table 12-5. Level 2 Analysis

Level 2 Analysis Example	
Determine Storm Tide Elevation	**Obtain appropriate FEMA flood map elevations for study area** FEMA FIS Report Summary of Stillwater Elevations of 12.5 ft for 50-year design storm tide Zone AE BFE shown on Flood Insurance Rate Map includes wave effects and runup and should not be used for stillwater elevation, reference the FIS for stillwater elevations
Determine Relative Sea Level Rise Rate	**Obtain relative SLR rates for project life cycle** Located in Orange County, Region 1; assumed 75-year design life for bridge starting in 2020; relative SLR for 2095 is 5.15 ft per Section 2 guidance
Interpolate Project Specific Relative Sea Level Rise	• the 5.15 ft for 2095 minus 0.8 ft from 2020; SLR of 4.35 ft for project life
Calculate Stillwater Elevation	**Add the result of Step 3 (relative SLR) to the elevations obtained in Step 1 (storm tide)** • 50-year: 12.5 ft plus 4.35 ft gives a stillwater elevation of 16.85 ft
Obtain Ground Data	**Obtain appropriate ground surface elevation maps for study area** LiDAR and bathymetry from TNRIS show an average elevation of 4 ft; ground elevation should be confirmed with plans, survey, or bathymetry.
Determine Stillwater Depth	**Subtract ground elevations (Step 5) from values found in Step 4 to obtain flood depth** • 50-year: 16.85 ft minus 4 ft equals a depth of 12.85 ft
Determine Maximum Wave Height	**Use a 1D wave model** Utilizing stillwater data obtained previously and local historical wind data from a nearby NOAA station, develop a wave analysis using a 1D wave model such as Coastal Engineering Design and Analysis System (CEDAS) or through spreadsheet-based calculations Model inputs include water depth, wind speed and duration, fetch length, typical temperature, and project latitude The resulting datasets should be statistically evaluated to determine a 50-year return period value for wave height and period (this example analysis yielded a 50-year wave height of 3.81 feet and a wave period of 3.82 seconds).
Determine Wave Crest Elevation (Design Elevation w/out Freeboard)	Multiply maximum wave height (Step 7) by 0.75, add to Step 4 for wave crest elevations; 75% of the wave height being above the stillwater elevation is a standard estimate for basic wave calculations (HEC-25) • 50-year: 3.81 ft times 0.75 plus 16.85 equals 19.71 ft Significant discrepancies between these values and historical storm observations; if the discrepancies are large, a Level 2 analysis may be required
Determine Bridge Design Elevation (Low Chord)	**Determine bridge low chord elevation^** Per TxDOT practice, a freeboard of 2 ft is added to the 50-year design wave crest elevation • 12.92 ft plus 2.0 ft equals 14.92 ft
^ Application of wave effects, relative SLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.	

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Table 12-6. Level 3 Analysis

Level 3 Analysis Example	
Determine Storm Tide Elevation	**Select, develop, and prepare appropriate numerical modeling tools** 2D and 3D hydrodynamic, wave, storm surge, and morpho-logic models (e.g., ADCIRC+SWAN, Delft3D, MIKE3) Calculations required for design could determine whether a 2D or 3D model is required If scour is expected to be critical, a 3D model may be necessary to account for complex current patterns at the structure Validate and/or calibrate the models through hindcast simulations and analysis Review past storms for data to use in calibrating the chosen model. *Elevations will be determined concurrently with the Design Wave Crest Elevation step*
Determine Relative Sea Level Rise Rate	**Obtain relative SLR rates for project life cycle** Located in Galveston County, Region 1; assumed 75-year design life for bridge starting in 2020; relative SLR for 2095 is 5.15 ft per Section 2 guidance
Interpolate Project Specific Relative Sea Level Rise	• the 5.15 ft for 2095 minus 0.8 ft from 2020; SLR of 4.35 ft for project life
Calculate Stillwater Elevation	*Elevations will be determined concurrently with the Design Wave Crest Elevation step*
Obtain Ground Data	**Obtain appropriate ground surface elevation maps for study area** An integrated LiDAR and bathymetry based digital elevation model will be necessary to utilize the model selected in at the beginning of the process
Determine Stillwater Depth	*Calculations will be internal to whatever model is chosen*
Determine Maximum Wave Height	*Elevations will be determined concurrently with the Design Wave Crest Elevation step*
Determine Wave Crest Elevation (Design Elevation w/out Freeboard)	Wave crest elevations, flow velocities, and others can be extracted from the model results Exceedance probabilities can be derived from the model output
Determine Bridge Design Elevation (Low Chord)	**Determine bridge low chord elevation^** Per TxDOT practice, a freeboard of 1 ft is added to the 50-year design wave crest elevation determined through Level 3 analysis Where debris flow is a concern, it is customary to use 3 ft for freeboard
^ Application of wave effects, relative SLR, and freeboard should be evaluated for each project independently. These calculations can be completed removing individual components as appropriate.	

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The TxDOT Geotechnical Manual provides more information on calculating scour. Note that results for additional return periods may be necessary in these calculations.

12.19 EROSION

When water moves through a channel, over land, or across subsurface terrain, aggradation, degradation, or a combination of both will occur due to instabilities between flow rate, slope, sediment load, and sediment size. The disequilibrium between these parameters can cause scour at various scales.

At a macro scale, the disequilibrium can cause entire channels to alter course or coastlines to shift in or out due to longshore currents. At a micro scale, constrictions in flow can cause increases in velocity and thus scour potential. These constrictions are common at bridge piers or other obstructions located within the flow path.

Examples of scour mechanisms at various scales are as follows:

- General Mechanisms
    - Scour around piers and abutments
    - Scour at highway embankment toe
    - Scour of embankment due to overtopping
    - Vertical degradation of streambeds
    - Horizontal migration of flow channels
    - Scour impacts from debris flow or debris clogging

- Coastal Specific Mechanisms
    - Infrastructure damage from wave crest attack
    - Damage from weir-like flow over linear infrastructure
    - Shoreline degradation leading to roadway undercutting

Structures in riverine environments usually only require scour calculations in a single direction as rivers tend not to reverse direction. However, coastal scour can occur both on the attack and the recession.

To assess local geology and thus the likelihood of scour at a project, it is necessary to perform a geotechnical investigation. Depending on the method of scour analysis, the investigation can include taking soil borings or surface samples. Much of the equations and other guidance is based on empirical data and field observations. Refer to the TxDOT Geotechnical Manual for details on the process.

Guidance for evaluating scour in tidal channels is outlined in the FHWA HEC-18, HEC-20, HEC-23, and HEC-25 publications and the USACE Coastal Engineering Manual.

DRAFT

In most cases, scour is not simple to quantify without assumptions, direct measurements, and hydrologic modeling. Three levels of analysis complexity apply to scour calculations.

Scour Level 1 Analysis – Qualitatively evaluate the long-term stability of the project site using the approach outlined in FHWA's HEC-18. Gather estimates of tidal magnitude, flow conditions, and storm conditions. This level of analysis is appropriate for culverts, generalized bridge opening sizing, and high-level inspections of bridge scour potential. 1D steady state modeling is warranted. This level of analysis should only be used for a project's planning phase and not for detailed project conditions.

Scour Level 2 Analysis – Quantitatively assess the velocity, depths, and flows at the project site. Using this data, the tidal prism method can be used to estimate flux of water between high and low tides at the project site. These can be used to generate scour rates based on that can be used for a wide variety of bridges and roads. 1D steady state modeling or design crest wave height determinations are sufficient.

Scour Level 3 Analysis – As complexity of obstructions and local terrain increases, the need for a Level 3 analysis increases. This can take the form of 1-dimensional unsteady state modeling to advanced 2D modeling. In rare cases where the project calls for it, physical models are recommended. Use FHWA's HEC-25 as a resource for approaches to Scour Level 3 analyses.

12.19.1 Scour Mitigation

FHWA's HEC-25 document recommends a five-part approach for determining and mitigating scour impacts. These steps are generally: manage and maintain, increase redundancy, relocate, accommodate, protect. See the summary of this approach in Table 15-11 in TxDOT HDM for more detail.

Reference HEC-23 guidance for further information related to scour mitigation.

12.19.2 Overwashing

Waves can bring water and sediment over coastal roadways when roads are built at-grade and close to the coastline. Overwashing can damage coastal roads in three ways:

a. Direct Wave Attack – If waves break at the seaward toe of the roadway, the breaking energy may, over time, undercut the roadbed, damage the road surface, or likely both.

b. Weir Flow – As water flows across the roadway in either direction, the roadway acts as a broad crested weir. As water flows over a weir crest (the edge of pavement in this case), it may transition from sub-critical flow to supercritical flow. This transition has erosive power that can eat away at whichever edge of pavement is acting as the weir crest. Sometimes, this water directly falls into the accumulated water on the downstream side of the weir edge. This repeated impact can cause scour. If the roadway is elevated on a berm however, the supercritical flow can scour the berm slope. As this water reaches the toe of the berm, this water may

DRAFT

undergo an additional change from supercritical back to subcritical. The associated hydraulic jump can further scour out the berm of the roadway.

c. Parallel Flow – As storms recede, the movement of the water may no longer defined by the intended design of the roadway, but by whatever damage the storm caused. As such, preferential pathways can form parallel to the roadway if new low spots or washouts have been created by the damage. These preferential pathways can scour out channels and undercut the roadbed.

The following table outlines potential mitigation solutions paired with the type of damage caused by overwashing.

Table 12-7. Mitigation Measures for Overwashing

Mitigation Measures	
Relocation – relocate roadway to a location and elevation that will allow incoming waves to bury and thus protect the roadway	Protects against all the above attack vectors as it "hides" the roadway from the wave front and from any hydraulic transitions.
Dune Construction – construct sand barriers adjacent to the roadway if relocation if not possible	Same mechanism as above.
Coastal Armoring – using engineered solutions to reinforce the stability of the coastline	Protects against all attack vectors as armoring methods steal or redirect momentum from incoming waves thus decreasing runup, wave heights, and the energy released when waves break.

12.19.3 Coastal Armoring Applicability

Three types of traditional armoring exist in a coastal context. Bulkheads, revetments, and seawalls have been proven effective over many years. The decision whether to implement any of these methods is largely based on the relationship between wave height and fetch length. The following table outlines general rules for when to apply each method.

Table 12-8. Types of Coastal Armoring

	What is it?	What does it do?
Bulkheads	Concrete, steel, or gabion walls along the length of the coast.	These act as both retaining walls and energy deflection/absorption devices for short-fetch, low wave height conditions in small inlets.
Revetments	Layers of engineered or natural riprap placed on the sloped coastline surface.	These absorb incoming wave energy so that the incoming water has less potential to scour away the protected soil. This method is suitable for intermediate-fetch lengths and wave heights found in bays or lakes.
Seawalls	A reinforced concrete or steel wa tall enough to protect against seaborne waves.	These work by deflecting the incoming energy back out into the water body. These structures can be designed to withstand incoming energy from long-fetch, high wave height conditions like the Gulf of Mexico.

DRAFT

While the above methods have a long history of use and the constructions of which is well understood, they are not always applicable and might even exacerbate the erosion problems at your project site. This is partly because these structures are designed to be rigid and inflexible, deflecting energy in ways that could potentially induce scour issues elsewhere. Soft armoring methods, whether used by themselves or in conjunction with traditional methods, can reduce this effect by being flexible enough to dynamically respond to future changes along the coastline.

12.20 SHORELINE CHANGE

In riverine environments, the rapidly moving water can cause degradation and aggradation of a channel on both micro and macro scales. In a micro-scale, scour can cause degradation at bridges. On a macro-scale, thalweg realignment can result in migration of the entire flowing body. Coastal degradation can also be examined on both scales. The macro scale degradation or aggradation of shorelines is generally called shoreline change. These shifts in shoreline are usually slow and are measured over lengths of time on the order of years. It should be noted that sudden, high-energy events can induce significant shoreline change even over short periods of time.

As noted in this chapter before, longshore currents along the Texas coast flow towards the northern stretch of Padre Island within Kenedy County. This slow process of shoreline change is depleting the south and north coasts and building the central Texas coast over a long period of time.

Since shoreline change is normally a macro, long-term issue, tracking the process becomes an exercise in obtaining data from across the region as well as diving into historical documentation. Long-term shoreline change rate estimates could include data from all the way back in the 1800s. If a long-term shoreline change rate has not been established for your location, a rate can be calculated via analysis of old maps and historical aerial imagery. Shoreline change rates have not been established for much of the Texas coastline.

Numerical models have been developed to estimate shoreline change. The choice between complexity of model should be based on an Analysis Level approach. Three levels of analysis complexity apply to shoreline change estimations.

a. Shoreline Change Level 1 Analysis – Evaluate historical imagery. Use a linear regression assumption to extrapolate shoreline position at a relevant future date unless engineering judgement says otherwise.

b. Shoreline Change Level 2 Analysis – Evaluate the beach profile with both topography and bathymetry in mind. Establish stillwater elevations and wave heights. Models such as CSHORE, CHAMPS, EDUNE, and SBEACH make use of these to generate shoreline change estimates.

c. Shoreline Change Level 3 Analysis – Evaluate pre- and post-storm beach profiles. Establish time series data for stillwater elevations and wave behavior. Using these (and more) inputs, models such as XBEACH, CMC, MIKE21, and DELFT3D can provide detailed shoreline change estimates.

DRAFT

The level of analysis used for shoreline change estimates needs to be dependent on the project. For projects involving rural locations where human infrastructure is minimal, using a linear estimate based on aerials is reasonable (Level 1). However, if the project area involves a highly built environment, an advanced (Level 3) modeling effort might be warranted.

The Texas Natural Resources Information System (TNRIS) maintains historic aerial photograph of the Corpus Christi area as early as 1979.

12.21 NATURAL COASTAL BARRIERS

In addition to actively reinforcing the coastline, passively using existing natural coastal barriers can be a simple and inexpensive option. Coastal barriers are a crucial component of the coastal ecosystem and help protect the coast during hurricanes and tropical storms.

The Coastal Barrier Resources Act (CBRA) was enacted in 1982 to remove federal incentives to develop or modify coastal barriers, and designated boundaries for the Coastal Barrier Resource System (CBRS). The CBRS comprised of System Units and Otherwise Protected Areas. System Units are designated areas of coastal barriers that are relatively undeveloped and are not eligible for financial assistance, flood insurance, or most new federal funding. Otherwise Protected Areas (OPAs) are portions of coastal barriers that are used primarily for natural resource protection, conservation, or recreation and are typically owned by federal, state, or local governments or nonprofit organizations. This includes national wildlife refuges, state and national parks, and local or private conservation areas.

Restrictions include but are not limited to the construction, purchase, or substantial improvement of any residence, building or structure, construction or purchase of any road, airport, or boat landing facility, flood control projects, dredging, and shoreline stabilization or erosion control (other than in cases of emergency). Some activities are exempt from CBRA restrictions, such as projects that include emergency assistance, military activities essential to national security, exploration and extraction of energy resources, and maintenance of existing federal navigation channels. Federal agencies are not prevented from issuing permits or conducting environmental studies under the CBRA. Additionally, the CBRA does not prohibit development within CBRS areas if fully funded by private developers or non-federal parties. Financial assistance prohibited by the CBRA includes any type of loan, grant, guaranty, insurance, payment, rebate, subsidy, or any other form of direct or indirect federal assistance. The CBRA does not restrict the use of state, local, or private funds within the CBRS.

There is no federal mandate for realtors or local officials to inform buyers or property owners that a property is within the CBRS boundary, and since 2018 CBRS boundaries are no longer depicted on FEMA Flood Insurance Rate Maps (FIRMs). The U.S. Fish & Wildlife Services website maintains an online Coastal Barrier Resources System Mapper that digitally maps the Coastal Barrier Resource System boundaries and can be used to determine if a property is located within the CBRS. The website is as follows: https://www.fws.gov/program/coastal-barrier-resources-act/maps-and-data.

DRAFT

12.22 TOPOGRAPHY AND BATHYMETRY

Topography and bathymetry both refer to the distribution of terrain elevations across an area, topography referring to on-shore and bathymetry referring to off-shore. Technically, bathymetry refers to the depths below a sea level datum, but that can easily be converted to an elevation. These datasets are essential to determining data inputs for Level 1 and Level 2 analyses.

For Level 3 analyses, it may be worthwhile to generate new data in and around the project site. New LiDAR datasets can be flown via drone or by pilot. New bathymetry can be obtained by operating a continuous depth sounder or acoustic profiler behind a boat. The following needs to be considered when using new data or combining datasets:

Date of Collection – data that is too old may not be relevant to your project (infrastructure may exist but be missing from the LiDAR due to recent construction)

Metadata – it is important to keep a record of the metadata involved in each dataset used in a project

The primary source of terrain information is via the Texas Geographic Information Office (TxGIO), formerly referred to as TNRIS. This organization aggregates spatial data from across the state and makes it available to the public. Visit the TxGIO website for the latest LiDAR and bathymetry data.

12.23 DESIGN ELEMENTS

Coastal structures need to be built with the objective of minimizing impact from nearshore processes. Several methods of protection are available depending on the local, site-specific requirements.

12.23.1 Roadway Design Considerations

Before considering any special design elements, attempt to align the roadway in such a way that avoids nearshore processes. If nearshore processes cannot be avoided, make sure that the design elevation is sufficient to avoid overtopping or wave impact. If the roadway alignment cannot be raised enough, techniques to avoid overwash damage should be applied. Embankment protections should be considered at any location where wave action or storm surge can impact the structure. Embankment protections can take the form of bulkheads, revetments, and seawalls. Specifics on the design of these protections can be found in the USACE Coastal Engineering Manual.

Coordinate with the City Public Works Department and TxDOT to identify evacuation corridors and any further design considerations that may be required.

12.23.2 Bridge Design Considerations

Bridge design in coastal environments begins as it does in upland environments, by establishing a low chord. Span, abutment, and piers are designed as they otherwise would be, keeping in mind the additional forces inherent in coastal storm events. Once

DRAFT

designed, the structure needs to be evaluated for impacts to nearshore processes. If the bridge project is in a transitional environment between riverine and coastal, the cumulative loadings from each need to be considered. Scour protection should be considered at the bridge piers and abutments. Scour protection usually takes the form of concrete riprap, stone, or geotextile beds anchoring the pier into the terrain surface. Specifics on the design of scour protection for bridge components can be found in the HEC-25 and USACE CEM documents. Bridge piers and abutments are also vulnerable to both hydrostatic and hydrodynamic forces that need to be accounted for in the design. Upsizing pier diameters can lead to a narrowing of the flow area. Narrowing the flow area can lead to increased flow velocity which in turn can lead to scour issues.

12.24 CONSTRUCTION MATERIALS

Materials used in coastal environments should be chosen specifically for their resistance to the effects of saltwater, as well as uplift/drawdown from storm event surges and retreats.

12.24.1 Saltwater

The coastal environment is antagonistic to infrastructure. Prolonged exposure to salt water by way of surface water or moisture suspended in air can corrode metal and concrete, can break down wood, and can even short electrical equipment.

12.24.2 Transportation Infrastructure

Infrastructure built in salty environments must be constructed using appropriate materials. USACE recommends marine grade steel be used in these areas.

For Portland Cement Concrete (PCC) based infrastructure, the primary corrodible material is steel reinforcement. To prevent oxidation and subsequent spalling of the concrete, plastic fiber reinforcement might be considered, depending on application. Reducing the permeability of the cement by including fly ash, blast furnace slag, silica fume, or concrete fines can help to extend the life of both the concrete and its reinforcement. Asphalt surfaces are not generally corrodible by salt exposure, but efforts can be made to increase their lifespan. Using soft asphalt binders and a low porosity bitumen-aggregate mixtures can help with this. Engineer must obtain City approval for concrete design for pavement and flatwork within the ROW or that will be maintained by the City

A more thorough treatment of transportation infrastructure construction in a coastal environment can be found through the following sources:

American Concrete Institute (ACI) – ACI 357R-84, Guide to the Design and Construction of Fixed Offshore Concrete Structures (Chapter 2)

The Aberdeen Group – Designing Concrete for Exposure to Seawater, Bruce A. Suprenant, Publication #C910873, 1991

DRAFT

FHWA – Techniques for Reducing Moisture Damage in Asphalt Mixtures, FHWA/TX-85/68+523+9F, November 1984

FHWA – Advanced High-Performance Materials for Highway Applications: A Report on the State of Technology, FHWAA-HIF-10-002, October 2010

USACE New Orleans District – Hurricane and Storm Damage Risk Reduction System Design Guidelines, Chapter 5

12.24.3 Building Construction

The risk of damage to household systems can be lowered by using components that have been approved for use in coastal environments. These can use special coatings and seals to prevent contact between the corrosive environment and the material.

A more thorough treatment of general construction in a coastal environment can be found through the following sources:

USACE – Coastal Engineering Manual

FEMA – Home Builder's Guide to Coastal Construction, FEMA P-499, December 2010

Note that on top of the guidelines outlined in the FEMA coastal construction document, there may be local floodplain ordinances that must be followed.

12.24.4 Surge and Drawdown Protection

As storms surge and subsequently draw down, the constructed environment is vulnerable to consistent bulk flow of water. With depth, velocities do not need to be high for bulk water to destroy buildings and the human-made environment. It is imperative for a building's insurability, a building's safety, and a building's resident's safety to adhere construction and maintenance recommendations in the documents presented in the previous section.

12.25 BUILDING CONSTRUCTION CONSIDERATIONS

Building design and construction equipment and materials are all important components to consider when constructing in coastal areas. FEMA's Coastal Construction Manual includes common observations of design flaws of coastal buildings based on historical storms, and recommendations for best practices.

Risks inherent to the coastal community include coastal storms and hurricanes, high winds, erosion, storm surge, wave action, high velocity flows, flooding, and sea-level rise. These hazards can cause structural damage and can gradually weaken structural integrity over time. Exposure to long term erosion and multiple coastal storms can cause flooding risks and other hazards to increase with time. Although a structure may "survive" one storm, it may deteriorate or weaken

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and become vulnerable to future storms. However, following proper practices and building codes can reduce the risks associated with building in a coastal zone.

Construction close to a shoreline is inherently vulnerable to wave, flood, and erosion risks. Margins of safety included in any design are at risk of being eliminated by those same forces. Consequently, coastal construction often requires rebuilding, redesign, reassessment, or removal. Even the best constructed buildings are made vulnerable if not sited properly. The presence of buildings in a coastal environment can lead to the concentration of flow pathways. Concentrated flow pathways result in increased erosion and scour in any environment, but on the coast, that process occurs faster.

As the salty air and water corrode and decay structural elements of a building over time, the price of deferred maintenance increases. As damage occurs, it becomes easier and easier for more serious damage to occur. If maintenance is deferred to the aftermath of a storm event, the total damage can easily be too much for the homeowner to handle. Refer to the City's Flood Hazard Prevention Code for information related to Substantial Damages and Substantial Improvement (SD/SI).

Storm surge and wave action can subject to debris impact, scour, high velocity flow, and/or significant hydrodynamic loads. Refer to FEMA's Coastal Construction Manual when considering building foundation types in Coastal A and V Zones.

It is also important to keep any electrical equipment and machinery elevated to prevent damage to the electronic components and decrease the possibility of electrocution during flood events. Refer to the City's Flood Hazard Prevention Code for specific elevation requirements.

12.26 ADDITIONAL COASTAL INFRASTRUCTURE CONSIDERATIONS

Stakeholder assessments are an important part of the early design process. This assessment may reveal additional design criteria for the project's design. A non-exhaustive list of potential stakeholder needs are as follows:

- Local or tourism-centric recreational activities
- Waterway users, specifically vessels, will require the bridge be elevated above a certain maximum vessel height
- Archaeological or historic sites along the coast are vulnerable to any changes in sediment aggradation/degradation patterns caused by a project
- Endangered species are vulnerable to habitat loss or habitat shift due to changes in sediment aggradation/degradation patterns

12.26.1 Building Codes and Standards

Building codes continue to evolve based on historical data and updated information, and lead to improved performance during storm events for structures that adhere to these upgraded standards. For more in-depth and further guidance regarding construction of buildings in coastal zones, reference the following resources in addition to the City's accepted building codes:

DRAFT

- Standard ASCE 24 Flood Resistant Design and Construction, published by the American Society of Civil Engineers (ASCE)
- Title 44 of the Code of Federal Regulations (CFR) Part 60 – NFIP regulations for enclosures and flood openings
- NFIP Technical Bulletin Series 1-11
- Subdivision Design in Flood Hazard Areas (Morris 1997), APA Planning Advisory Service Report Number 473
- Hazard Mitigation: Integrating Best Practices into Planning (Schwab 2010), APA Planning Advisory Service Report Number 560

12.27 CRITICAL FACILITIES

Critical facilities (or essential facilities) are public or private infrastructure that are critical to protecting the safety or health of the community, or that provide vital services to the community. These facilities may be especially important for the recovery of the community following storm events. These facilities may include but are not limited to:

- Hospitals
- Fire stations
- Police stations
- Nursing Homes
- Education facilities including schools, daycares, and universities
- Evacuation routes
- Airports and aviation facilities
- Emergency operation centers
- Emergency shelters
- Drinking water supply facilities
- Wastewater treatment facilities
- Power stations
- Communications facilities
- Hazardous waste facilities

Disruption of these facilities or the services they provide could have serious consequences on their community, impacting recovery efforts such as access to damaged areas, impairment of search and rescue teams, or inability to provide emergency medical care. For these facilities or infrastructures, special consideration should be taken during design, construction, operation, and maintenance to ensure they are well protected during flood or storm events.

For more information regarding construction and improvement of critical facilities, refer to ASCE 7, *Minimum Design Loads for Buildings and Other Structures* and FEMA 543, *Design Guide for Improving Critical Facility Safety from Flooding and High Winds*.

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12.28 POLICIES AND REGULATION RELEVANT TO COASTAL CONSTRUCTION

12.28.1 Federal

Rivers and Harbors Act of 1899 – restricts construction, excavation, or filling in navigable waters of the United States that would obstruct navigation, alter its course, or change its capacity

Migratory Bird Treaty Act of 1918 – protects migratory birds by prohibiting the hunting, capturing, killing, or selling of these birds without proper permits or authorization

National Environmental Policy Act of 1969 – establishes as national policy, the goal of environmental protection; introduces the concept of an Environmental Impact Study to assess potential environmental impacts of most federal projects

National Flood Insurance Act of 1968 – established the NFIP with the goal of reducing flood losses through local management of floodplains; prompted the new program to develop baseline ordinances to be met or surpassed locally for participating communities

Marine-Mammal Protection Act of 1972 – safeguards marine mammals by prohibiting their harassment, hunting, capturing, or killing, and promoting their conservation and protection in their natural habitats

Magnuson-Stevens Fishery Conservation and Management Act of 1976 – regulates the management and conservation of marine fisheries, establishes regional fishery management councils, and aims to prevent overfishing and rebuild overfished stocks to maintain sustainable fish populations

Executive Order 11988 (May 24, 1977) – requires federal agencies to avoid impacting the floodplains that they may choose to disturb when constructing federal facilities

Clean Water Act of 1972 – aims to restore and maintain the chemical, physical, and biological integrity of the nation's waters by regulating pollutant discharge into water bodies and setting water quality standards

Section 401 –	certification under this section is required prior to the issuance of any permits; effluent from the project at all stages is required to meet state water quality standards
Section 402 –	establishes a nationwide permit system to regulate pollutant sources emanating from a project site
Section 404 –	establishes a permit system to regulate discharge of dredged fill into rivers, streams, and wetlands

DRAFT

Coastal Barrier Resources Act of 1982 – restricts federal financial assistance and development in designated coastal barrier areas to minimize the risk of damage from storms, protect natural resources, and disincentivize construction along vulnerable coastal zones; establishes the Coastal Barrier Resources System to identify critical coastal barrier resources

12.28.2 State

Texas Water Code Chapter 11

Section 11.021 –	surface water is owned by the state and access to it is controlled by a system of water rights administered by the state
Section 11.086 –	continuing to or choosing to divert or obstruct natural surface water flows in a manner that is injurious to another's' property is unlawful and remedies at law exist

Texas Water Code Chapter 16 Subchapter I – establishes the framework for how Texas will conform with the NFIP

Title 30 Texas Administrative Code Rule 15.54(e) – those interested in discharging drainage onto or across a state highway right of way where drainage does not already exist, must obtain approval from TxDOT

The lists of applicable federal and state regulations are not necessarily exhaustive.

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Chapter 13
MISCELLANEOUS CRITERIA

13.1 DRAINAGE RIGHT-OF-WAY & EASEMENTS

The following criteria apply for drainage rights-of-way and easements.

a. Where not specifically discussed in this Drainage Design Manual, minimum required drainage ROW and easements widths shall be per the UDC,

b. Minimum required drainage ROW/easement width for storm water channels shall be inclusive of the top width plus required maintenance strips.

c. Minimum required easements for detention ponds shall be inclusive of the limits of the pond, berms, and associated maintenance strips.

d. Access to maintenance strips from ROW or other easements shall be included in an easement or ROW.

e. Easements and ROW for storm water facilities must be wide enough to contain the storm water infrastructure that would be needed to convey runoff assuming the upstream watershed is fully developed.

f. Extreme event overflow corridor easements shall be dedicated and may include streets, channels, and open spaces.

13.2 FLOODPLAIN DEVELOPMENT AND FINISHED FLOOR ELEVATION

The following criteria apply for development in floodways/floodplains:

a. Development in delineated floodways or floodplains must meet the requirements established by FEMA, plus any higher standards prescribed by the City.

b. New construction in FEMA flood hazard areas must have a minimum first floor elevation for habitable living space of at least 12” above the Base Flood Elevation (BFE).

c. New construction outside FEMA flood hazard areas must have a minimum first floor elevation for habitable living space at least 18” above the lowest adjacent top of curb, crown of road, or top of bank of any adjacent channel or detention pond.

13.3 LOT GRADING AND DRAINAGE

The following criteria apply for lot grading and drainage:

a. Lot grading shall be from back to front toward the street, swale or inlet.

DRAFT

b. Lot grading shall be at a minimum slope of 1% for unpaved areas.

c. Lot grading with paved areas shall be minimum:

- 1% for HMAC pavement
- 0.3% for concrete pavement and flatwork

d. Surface drainage from one lot across another lot in a residential subdivision shall not be allowed without establishment of a drainage easement.

13.4 MAINTENANCE

Provisions for adequate maintenance must be made in the design of all drainage facilities. Sufficient right-of-way must be set aside, slopes must be kept at or below maximum values, and slope treatments must be properly completed. Access to drainage facilities must not be impeded.

a. For existing systems, the City may secure additional drainage right-of-way and/or drainage easement width in order to improve access and enhance operations and maintenance activities, when acquisition is deemed necessary and feasible.

b. Open channels located in ROW dedicated to the City of Corpus Christi will be maintained by the City.

c. Open channels located in easements within private property will be maintained by the property owner unless otherwise agreed to by the City.

d. Detention ponds will be maintained by the property owner.

e. Access to channel maintenance strips must be unimpeded, and multiple access points should be provided wherever possible.

f. Conflicts from fences, power poles and utility appurtenances shall be minimized. If the City determines that joint occupancy utilities located within a drainage right-of-way or drainage easement interfere with storm water operations and maintenance activities, the utilities are required to relocate at no cost to the City.

g. Special attention shall be given to providing maintenance access along and behind bridge and culvert guardrails from the roadway down to the channel maintenance strip.

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APPENDIX A

ACRONYMS & ABBREVIATIONS

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ACRONYMS & ABBREVIATIONS

ASTM:	American Society for Testing and Materials	**MS4:**	Municipal Separate Storm Sewer Systems
BFE:	Base Flood Elevation	**NFIP:**	National Flood Insurance Program
BMP:	Best Management Practice	**NOAA:**	National Oceanic & Atmospheric Administration
CDS:	Continuous Deflective Separation	**NPDES:**	National Pollutant Discharge Elimination System
CFR:	Code of Federal Regulation	**NRCS:**	Natural Resources Conservation Service
CIP:	Capital Improvement Program	**NSWP:**	National Storm Water Program
CMP:	Corrugated Metal Pipe	**RCP:**	Reinforced Concrete Pipe
CWA:	Clean Water Act	**ROW:**	Right-of-Way
DCM:	Drainage Criteria Manual	**SCS:**	Soil Conservation Services
DDM:	Drainage Design Manual	**SSURGO:**	Soil Survey Geographic Database
DEMs:	Digital Elevation Models	**SWICA:**	Storm Water Infrastructure Contract Agreement
EPA:	Environmental Protection Agency	**SWMP:**	Storm Water Master Plan
ETJ:	Extraterritorial Jurisdiction	**SWPPP or SW3P:**	Storm Water Pollution Prevention Plan
FEMA:	Federal Emergency Management Agency	**SWQM:**	Storm Water Quality Management
FHWA:	Federal Highway Administration	**TCEQ:**	Texas Commission on Environmental Quality
FIRM:	Flood Insurance Rate Map	**TMDL:**	Total Maximum Daily Load
FIS:	Flood Insurance Study	**TNRCC:**	Texas Natural Resource Conservation Commission
GIS:	Geographic Information System	**TOB:**	Top of Bank
HEC-HMS:	Hydrologic Engineering Center - Hydrologic Modeling System	**TPDES:**	Texas Pollutant Discharge Elimination System
HEC-RAS:	Hydrologic Engineering Center - River Analysis System	**TxDOT:**	Texas Department of Transportation
HGL:	Hydraulic Grade Line	**TxGIO:**	Texas Geographic Information Office
HRT:	Hydraulic Residence Time	**UIOC:**	Unimproved Open Channel
IDF:	Intensity-Duration-Frequency	**USACE:**	United States Army Corps of Engineers
IFA:	Ineffective Flow Areas	**USDA:**	United States Department of Agriculture
LIDAR:	Light Detection and Ranging	**USEPA:**	United States Environmental Protection Agency
LOP:	Level of Protection	**USGS:**	United States Geological Survey
LUI:	Land Use Intensity	**WSE:**	Water Surface Elevation

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APPENDIX B

GLOSSARY

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GLOSSARY OF KEY WORD DEFININITIONS

Adverse Impacts: Increases in downstream flows or elevations, upstream elevations, or elevations on adjacent properties.

Area Development Plan: A component of the City's Comprehensive Plan that give major consideration to land use issues and address allocation of services, facilities and other area-specific issues. Their purpose is to provide decision-makers with a guide to manage future development.

Backslope Interceptor Structure: A drainage structure within a backslope swale that conveys stormwater into a channel or detention basin.

Backslope Drainage Swale: A shallow swale within a maintenance strip that is designed to intercept overland flows and prevent erosion of side slopes.

Base Flood: A base flood is the national standard on which the floodplain management and insurance requirements of the National Flood Insurance Program (NFIP) are based. Special Flood Hazard Areas (SFHA's) are depicted on FEMA Flood Insurance Rate Maps (FIRM's) and are areas subject to inundation by the base flood having a one-percent or greater probability of being equaled or exceeded during any given year (also known as a 100- year flood event).

Base Flood Elevation (BFE):
An elevation of the water surface within a drainage corridor at which the storm water from a 100-year event is estimated to rise based upon a FEMA-approved hydrologic and hydraulic analysis of that corridor.

Benchmark: Data used as a base for comparative purposes with comparable data.

Benchmark Information: A description of the benchmark used to establish existing and proposed elevations in the project area, including the exact location, the elevation, and the source of the elevation.

Best Management Practices (BMP's): Means or methods that reduce pollutant loading to downstream elements. BMP's can be either structural controls or practices or non-structural controls and practices. Non-structural practices include but are not limited to inlet cleaning, street sweeping and detention pond maintenance.

Buffer Strip: Strip or area of vegetation used for removing sediment, organic matter, and other pollutants from storm water runoff.

Capital Improvement Program (CIP): A projected schedule of capital projects based on estimated costs and expected funding levels.

Channel: An open storm water conveyance facility with side slopes ranging from two to four units horizontally to one unit vertically.

Clean Water Act (CWA): Contains a number of provisions to restore and maintain the quality of the nation's water resources.

Concrete-Lined Channel: A channel that has been lined with concrete on the side slopes and bottom.

Conduit: A conduit is any open or closed device for conveying flowing water.

Conveyance: The ability of a channel or conduit to carry water in the downstream direction.

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Criteria: A standard or rule on which a judgment or decision is based.

Critical Depth: The depth of flow at which specific energy is at a minimum. At critical depth, the flow is neither subcritical nor supercritical.

Critical Facility: A facility that serves a critical function for the community. Examples include, but are not limited to water and wastewater treatment and conveyance systems, emergency operations facilities, and key telecommunication and electrical systems.

Cross-Sectional Area: The total area available to carry flow, measured at a vertical plane (cross-section) which cuts across a channel or conduit perpendicular to the direction of flow.

Design Capacity: The amount of water that a storm water facility is designed to manage usually expressed in cubic feet per second for flow and cubic feet or acre feet for detention.

Design Storm Event: The rainfall intensity upon which the drainage facility will be sized.

Detention or To Detain: To temporarily hold storm water in such a way as to regulate its rate of flow, either to limit downstream impacts or to provide time for natural processes to have a positive impact upon water quality.

Detention Basin: A storm water facility designed to capture and limit storm water flow (by releasing it at a reduced rate) in order to reduce downstream impacts or to treat storm water to improve its quality.

Developer: A proponent of a project that alters the natural state of the land upon which that project is to be built. A Developer can be a private individual, a landowner, a tenant, a business partnership, company or corporation, or a government entity or agency.

Discharge Calculations: Calculations specifying computed discharges at key locations, with comparisons of existing and proposed discharges where appropriate. Drainage areas, runoff coefficients, rainfall depths and intensities, infiltration loss parameters, unit hydrograph parameters, and other applicable hydrologic data shall be included and clearly documented. For computer applications, printouts shall be attached.

Drainage Basin: Any land area from which the runoff collects into a common point or receiving water. The area of land that drains water, sediment, and dissolved materials to a common outlet at some point along a stream channel. Also called a watershed.

Drainage Easement: Property right, which enables the City to install, operate and maintain minor storm water facilities. Although the land is still owned by others (fee simple) the City has proprietary right of use.

Drainage Map: A map which illustrates all drainage boundaries, flow directions, and computation points.

Drainage Right-of-Way: Property right, which enables the City to install, operate and maintain major storm water facilities. Right- of-way dedication may be a fee simple transaction.

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Earthen Channel: A man-made channel that is grass-lined on the bottom and/or side slopes.

Ecosystem: The interacting system of a biological community of animals, plants, and bacteria, with its non-living, interrelated physical and chemical environmental surroundings.

Elevation: "Elevation" means height above mean sea level. The vertical control system (benchmarks) referenced in the most current Flood Insurance Study must be used except in coastal areas where subsidence has occurred. Any future studies changing the Flood Insurance Rate Map (FIRM) which is referenced to a later re-leveling of the vertical control system must be used whenever a revised FIRM becomes effective.

Energy Dissipator: A structure or device used to decrease the energy of water (i.e., reduce velocity). The purpose of an energy dissipator is to reduce the potential for scour or erosion that would result from high velocity flow.

Engineer: A registered, professional engineer licensed to practice in the State of Texas.

Environmentally Sensitive Area Analysis: Delineation on the site map shall clearly identify environmentally sensitive areas. A description of temporary and permanent BMPs (structural and nonstructural) shall be provided. The report shall include the type and location of environmentally sensitive standard City details to be used on the project. If applicable, a statement shall be provided that asserts that no portion of the property resides in an environmentally sensitive area, nor will off-site environmentally sensitive areas be impacted by storm water pollution from the site.

Erosion: Removal and transportation of soil in a stream system (generally includes the processes of scour, transport, and deposition).

Erosion Protection: Used to prevent or minimize erosion due to excess velocities or unstable soils, including: interlocking concrete blocks, rip-rap, geotextile liners, or concrete lining.

Evaporation: Process of changing visible water into invisible vapor.

Evapotranspiration: The total evaporation of rainfall from all sources such as evaporation of precipitation intercepted by plant surfaces, evaporation of moisture from plants by transpiration, and evaporation of moisture from the soil surface.

Existing Conditions: Current watershed and channel conditions (prior to project).

Extraterritorial Jurisdiction (ETJ):
The region outside of the City limits, within which the City retains authority for enforcement of City ordinances, policies, codes and regulations. For the City of Corpus Christi, the ETJ is a line that is 5 miles beyond the City limits line.

Facility Layout Map: Plan, elevation, and cross-section views of drainage facilities such as detention basins, roadway culverts, bridges.

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Fascine: A long bundle of sticks bound together and used for such purposes as filling ditches and making revetments for riverbanks.

Feasible: Refers to whether a project approach is technically viable. Although consideration to the economic practicality is included, it is understood that when a project uses design criteria that is less than the desired level it will be less costly. Project economics alone is not sufficient reason to state that meeting the desired criteria is not feasible.

Federal Emergency Management Agency (FEMA): Federal agency which administers the National Flood Insurance Program.

Flood Insurance Rate Map (FIRM): A map created by the Federal Emergency Management Agency, under the National Flood Insurance Program, that delineates flood hazard areas.

Flood Insurance Study (FIS): A document containing the results of an examination, evaluation, and determination of flood hazards and, if appropriate, corresponding water surface elevations, mudslides and erosion hazards.

Floodplain: An area inundated by flood waters during or after a storm event of a specific magnitude.

Floodplain Maps: A ***Flood Insurance Rate Map*** showing the boundaries of the existing 100-year floodplain and floodway in the project area and a separate map which illustrates proposed changes in floodplain or floodway boundaries.

Floodway: Corridor of effective flow area that consists of the channel and any adjacent land needed to convey the 100-year base flood without cumulatively increasing the water surface elevation more than one-foot above the base flood elevation.

Flume: A channel lined with bituminous concrete, Portland cement concrete, or comparable non-erodible material placed to extend form the top of a slope to the bottom of a slope.

Forebay: A reservoir or pond situated at the intake of a detention basin to stabilize or regulate water levels.

Freeboard: Elevation difference between the top of bank and design water surface elevation.

Friction Loss: A loss in energy associated with friction between flowing water and the sides of a channel or conduit.

Froude Number (Fr): The dimensionless ratio of the inertial and gravitational forces of water.

Gabion: A basket or cage filled with rocks and used especially in building a support or abutment.

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Geographic Information System (GIS): A digital, electronic data and information storage and retrieval format tied to a geographical reference such as a map of a city.

Greenway or Greenbelt: A linear open space established along either a natural corridor, such as a riverfront or stream, or overland along a railroad right-of-way converted to recreational use, a canal, a scenic road, or other route.

Hydraulics or Hydraulic Analysis: Study of how the runoff volume reacting in time produces water surface elevations.

Hydraulic Calculations: Hydraulic calculations specifying the methods used in analyzing channels, storm sewers, and other hydraulic structures and providing a summary of the results obtained. Cross- section data, roughness coefficients, flow rates, and other data shall be clearly documented. For computer applications, a digital submittal containing input files for all hydraulic models and printouts shall be attached.

Hydraulic Grade Line (HGL): A hydraulic profile of the piezometric level of the water, representing the sum of the depth of flow and the pressure head. In open channel flow, the HGL is the water surface.

Hydraulic Jump: The rapid change in the depth of flow from a low stage to a high stage, resulting in an abrupt rise of water surface.

Hydraulic Radius: A parameter computed as the cross-sectional area divided by the wetted perimeter.

Hydrograph: A graph which relates rate of flow and time.

Hydrologic Cycle: The cycle experienced by water in its travel from the ocean, through evaporation and precipitation, percolation, runoff, and return to the ocean.

Hydrology: The study of the processes through which atmospheric moisture passes, between the time that it falls to the surface of the earth as rain and the time that it returns to the atmosphere.

Impervious Area: Land surfaces which do not allow (or minimally allow) the penetration of water. An increase in the amount of impervious area will increase the rate and volume of runoff from a given drainage basin.

In-Fill Development: The development of isolated, discontinuous, undeveloped tracts of land in areas where urban development has already taken place.

Infiltration: The process by which rainfall soaks into the ground.

Inlet: A structure that allows storm water to flow into a conveyance system. On-Grade Inlets are on a surface that slopes away from the opening, such that flow could bypass the inlet if clogged or at capacity. “Sump” or “Sag” Inlets are at low points.

Intensity: Measurement of rainfall depth per unit of time, usually in inches/hour.

Interception: Capture of raindrops by vegetation that prevents them from falling on the ground.

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Interceptor Structure: A structure, such as a post inlet, that captures overland or channelized flow and directs it into a receiving channel.

Jurisdictional Wetlands: An area that meets the criteria established by the *U.S. Army Corps of Engineers* for wetlands. Such areas come under the jurisdiction of the USACE for permitting certain actions such as dredge and fill operations.

Levee: An embankment alongside a river, stream or other water course that is used to prevent flooding.

Level of Protection (LOP): The level of flooding that a community decides is acceptable given a storm event of a certain frequency and intensity, balancing costs against property protection and convenience benefits.

Light Imaging Detection and Ranging (LIDAR): LIDAR produces highly-detailed ground elevation data and utilizes the projection of millions of laser signals to the ground from a specially-equipped aircraft. Using powerful software, the data from these LIDAR reflections is collected by measuring the time it takes for the aircraft to receive each of the millions of laser reflections. The resulting data is then combined and converted into an image that looks exactly like the terrain below, including buildings, trees, roadways, creeks and bayous.

Maintenance Strip: A dedicated strip of land along the top of bank of a channel or detention basin used to access the facility for maintenance.

Major Channel: Open channels and other pertinent drainage structures associated with open channels that serve drainage areas equal to or greater than 500 acres.

Major Storm Sewers: Inlets, laterals, and main storm sewer pipe or box systems that serve drainage areas more than 500 acres. These typically serve subdivisions, residential streets, collector streets, and arterial streets.

Manning's Equation: A mathematical formula which relates the velocity or rate of flow in a channel or conduit to the physical characteristics of the channel or conduit.

Maximum Allowable Discharge:
Maximum discharge allowed from a detention basin for a proposed development, which is generally limited to the existing conditions peak flow.

Minor Channel: Open channels and other pertinent drainage structures associated with open channels that serve drainage areas less than 200 acres.

Minor Loss: A loss in energy associated with changes in flow direction or velocity.

Minor Storm Sewers: Inlets, laterals, and main storm sewer pipe or box systems that serve drainage areas less than 200 acres. These typically serve subdivisions, residential streets, collector streets and arterial streets.

Mitigate: Measures taken to eliminate an adverse impact caused by an action.

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Model (Analysis): Approximations of the hydraulics and hydrology of a drainage basin based upon mathematical derivations of quantifiable relationships between various factors. These factors usually include, but are not limited to, area, slope, soils, drainage system characteristics, rainfall and land use.

MS4: **M**unicipal **S**eparate **S**torm **S**ewer **S**ystem – A conveyance or system that is owned or operated by a State, City, Town, Association, or other public body which is designed or used for collecting or conveying storm water.

MS4 Co-Permittees: City of Corpus Christi and TXDOT-Corpus Christi District, Corpus Christi Junior College, Port of Corpus Christi Authority, and Texas A&M University-Corpus Christi.

National Flood Insurance Program (NFIP): Created by U.S. Congress in 1968, it provides federally backed flood insurance that encourages communities to enact and enforce floodplain regulations.

National Pollutant Discharge Elimination System (NPDES): The national program for issuing permits and enforcing the pre- treatment requirements of the Clean Water Act.

National Storm Water Program (NSWP): A Federal government initiative directed by the U.S. E.P.A. with voluntary cooperation with authorized States and mandatory participation of local government agencies. This program regulates storm water discharges throughout the United States.

Natural Channels: Naturally occurring channels (not man-made), such as streams and rivers.

Natural Drainage Ways: See Natural Channels***.***

Non-Point Source: Diffuse pollution sources (i.e. without a single point of origin or not introduced into a receiving stream from a specific outlet). The pollutants are generally carried off the land by storm water. Common non-point sources are agriculture, forestry, urban, mining, construction, dams, channels, land disposal, saltwater intrusion, and city streets.

Nonstructural Controls: Pollution prevention behaviors and methods that do not include physical or structural controls; encouraged as the first step in water quality protection.

Normal Depth: Depth associated with uniform flow.

On-Grade Inlet: An inlet located on the street with a continuous slope past the inlet with water entering from one direction.

Outfall: Location where storm water leaves a given conveyance system. The ultimate outfall of a system is usually a receiving water.

Peak Flow: Maximum flow from a drainage area during a specific rainfall event.

Pervious Area: Land surfaces that allow the penetration of water. A decrease in pervious area will increase the rate and volume of runoff from a given drainage basin.

Pilot Channel: Small swale used to provide positive drainage in the bottom of detention basins or channels.

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Plotted Cross-Sections: Typical cross- sections of the subject stream for both existing and proposed conditions.

Plotted Stream Profile: A profile of the subject stream, which includes computed water surface profiles; existing and proposed flow-line profiles; the locations of existing and proposed bridges, culverts, and utility crossings; the locations of tributary confluences and Major storm sewer outfalls in or near the project area; and the locations of hydraulic structures such as dams, weirs, and drop structures.

Point Source: A stationary location or fixed facility from which pollutants are discharged; any single identifiable source of pollution such as a pipe or ditch.

Policy: A plan or course of action, as of a government, political party, or business, intended to influence and determine decisions, actions, and other matters

Pollutant: Generally, any substance introduced into the environment that adversely affects the usefulness of the resource.

Pollution: Generally, the presence of matter or energy whose nature, location, or quantity produces undesired environmental effects. Under the Clean Water Act, the term is defined as the human-made or human- induced alteration of the physical, biological, chemical, or radiological integrity of water.

Pollution Control Measures: The minimum structural and non-structural storm water pollution control practices required for site development projects.

Ponding Spread: The extent to which storm water will reach horizontally at a given vertical depth.

Probability: The chance, usually expressed in percent, that a storm event of a particular intensity and duration will occur in any given year. Equal to the reciprocal of the recurrence interval.

Proponent: An entity, public or private that initiates, promotes, and advocates the planning, design, and construction of a particular project. A proponent can be the City, another government agency, a developer, or other private citizen or group.

Proposed Conditions: Conditions after a project is implemented (future watershed and channel conditions).

Rainfall Frequency: The probability of a rainfall event of defined characteristics occurring in any given year (i.e., the 100- year event is equal to the rainfall intensity having a 1% probability of occurring or being exceeded in a given year).

Rainfall Intensity: The rate at which rainfall occurs, typically expressed in inches per hour.

Reach: A specified length of a stream or conveyance. Often a length of channel which is uniform in its discharge, depth, area, and slope.

Receiving Water: A body of water that receives and stores storm water flow. Typical bodies include, but are not limited to, lakes, creeks, bayous, rivers, bays and oceans.

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Rectangular Concrete Low-Flow Section: A concrete lining that covers the bottom of a channel and has vertical walls that extend partially up the side slopes. These can be incorporated into designs for earthen and concrete-lined channels. The low flow section may be used to provide flow capacity in areas where the availability of right-of-way is limited.

Recurrence Interval: The average period of time that will elapse between storms of a particular intensity and duration (equal to the reciprocal of the probability).

Redevelopment Project: A change in land use that alters the impervious cover from one type of development to either the same type or another type.

Retention or To Retain: To store storm water to prevent its discharge into receiving waters or to provide a storage facility for storm water where no outfall is available.

Retention Pond: A storm water facility from which the discharge from is limited to percolation, evaporation, and evapotranspiration.

Revetment: A facing to sustain an embankment.

Right-of-Way Map: A map which illustrates existing and proposed channel and utility rights-of-way and easements. Include both underground and overhead utilities and all drainage easements or ROW.

Rock Riprap: A layer of crushed concrete, loose rock, or aggregate placed over an erodible soil surface.

Roughness Coefficient: A number which represents the relative resistance to flow in a channel or conduit.

Runoff: The residual precipitation remaining after deduction or interception and evapotranspiration losses. It appears in surface channels, natural or man made, whose flow is perennial or intermittent. Classified by the path taken to a channel, runoff may be surface, subsurface or groundwater flow.

Sag: A low-lying point in the pavement that receives drainage.

Scour: The erosive action of running water in streams, which excavates and carries away material from the bed and banks. Scour may occur in both earth and solid rock material.

Shall: A directive or requirement.

Sheet Flow: Overland storm runoff that is not conveyed in a defined channel or conduit and is typically in excess of the capacity of the conduit or roadside ditch.

Should: An expectation.

Side Slope (SS): The horizontal to vertical ratio of the slope from top of bank to toe of slope. The minimum allowable SS is 4:1 for grass-lined channels in Corpus Christi.

Site Map: This is a detailed map of the project site which illustrates the type and extent of activities which are proposed to be completed. For new developments, a plat with all proposed streets, lot boundaries, etc. may be used to satisfy this requirement.

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Soils Report: A soils report shall be prepared by a qualified geotechnical Engineer, and shall identify the existing soil types and assess the suitability of the soil for the proposed activity. The soils report shall address erosion and slope stability in areas subject to the action of storm runoff.

Special Flood Hazard Areas: Those drainage corridors that have been designated by FEMA as 100-year floodplains.

Specific Energy: The sum of the piezometric head and the velocity head; total energy, with respect to the bottom of a conduit or channel as a datum.

Steady Flow: Discharge does not vary with time.

Storm Event: A single period of heavy rainfall that normally lasts from a few minutes to a few days.

Storm Water: Flow of water, which results from, & which occurs immediately after a rainfall event.

Storm Water Management Facility: A feature that collects, conveys, channels, holds, inhibits, or diverts the movement of storm water.

Storm Water Management Program: The City's overall strategy for managing storm water runoff conveyance and quality.

Structural Controls and Practices: Physical means and methods to control storm water runoff or to reduce any potential pollutants from being introduced into receiving water.

Subcritical: Flow regime in which gravitational forces control the rate of flow. The flow will have a low velocity and appear tranquil.

Sump: A sag from which water generally can not escape without overland passage via an extreme event corridor.

Supercritical: Flow regime in which inertial forces control the rate of flow, and the flow can be described as shooting and rapid.

Surcharge: The runoff in excess of the actual capacity of a storm water facility.

Swale: A very shallow open storm water conveyance facility.

Tailwater: The water into which an outfall discharges.

Time of Concentration: The time required for water to travel from the most remote point in a watershed to the point at which a peak flow rate or runoff hydrograph is to be computed.

Toe of Slope: Point where the side slope intersects with the channel or detention basin bottom.

Top of Bank: The high point along the bank of a channel or a detention basin.

Tributary: A stream which joins another stream or body of water.

Ultimate Conditions: Full development of an entire watershed, typically based on zoning, land use plans, or an assumed mixed use when no other data is available.

Uniform Flow: Flow with straight, parallel streamlines.

Unimproved Open Channel: A natural or manmade channel that, without additional capacity, geometric, surface material and/or

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excavation modifications, will not serve the conveyance needs of the service area.

Unit Hydrograph: A runoff hydrograph which represents the response of a watershed to 1 inch of runoff.

Unsteady Flow: Discharge varies with time at one or more points.

Vicinity Map: A map showing the project site with respect to recognizable landmarks in the vicinity. This could be a base map with the boundaries of a new development or the limits of an infrastructure improvement project indicated to mark the project location.

Watercourse: A path which water follows from the boundary of a watershed to the watershed outlet.

Water Quality: A term used to describe the chemical, physical, and biological characteristics of water, usually in respect to its suitability for a particular purpose.

Water Quality Criteria: Levels of water quality expected to render a body of water suitable for it's designated use. Criteria are based on specific levels of pollutants that would make the water harmful if used for drinking, swimming, fish production, farming, or industrial processes.

Water Quality Standards: State-adopted and EPA-approved ambient standards for water bodies. The standards prescribe the use of the water body and establish the water quality criteria that must be met to protect designated uses.

Watershed: The entire land area bounded by ridge lines that drains into a major river, stream or bay. A watershed may encompass numerous basins that ultimately combine at a common delivery point.

Water Surface Elevation: The level of the surface of the water.

Wetland: An area that is saturated by surface or groundwater with vegetation adapted for life under those soil conditions, such as swamps, bogs, fens, marshes and estuaries.

Wetted Perimeter: The total distance along a channel or conduit cross-section, which is in contact with water that is flowing in the channel or conduit.

Will: An expectation of performance that is generally accepted as being a responsibility.

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APPENDIX C

CHECKLISTS

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CHECKLIST C-1

STORM WATER QUALITY MANAGEMENT PLAN CHECKLIST
CITY OF CORPUS CHRISTI

A Storm Water Quality Management Plan (SWQMP) must be submitted at the time of preliminary plat submission; a final plat submission if no preliminary plat was submitted, or with an application for building permit, if not already submitted.

The Plan applies to all construction projects of 1 acre or greater and means an engineering report and analysis that provides:

☐ 1. Project narrative, including project background, hydrologic and hydraulic assumptions, floodplain impacts, existing conditions and proposed improvements, recommendations and conclusion.

☐ 2. Hydrologic calculations, including land use, time of concentration/lag times, peak flows, pond volumes, etc.

☐ 3. Hydraulic calculations, including inlet capacity, pipe/culvert/channel capacity, velocity, HGL, EGL, pond outfalls, dissipators, etc.

☐ 4. Maps and exhibits, including

   a. Location Map
   b. FEMA Floodplain Map
   c. Offsite and onsite drainage area maps
   d. Soils, land use, and topographic maps
   e. Drainage plan with existing and proposed storm water infrastructure and hydraulic calculations
   f. Detention pond plans and details
   g. Standard and project-specific details

☐ 5. Delineation of the route of runoff to ultimate outfall.

☐ 6. Identification of any Environmentally Sensitive Area that is on the site or would be sensitive to storm water pollution from the site.

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CHECKLIST C-2

STORM WATER POLLUTION PREVENTION PLAN CHECKLIST
CITY OF CORPUS CHRISTI

Storm Water Pollution Prevention Plans (***SWPPP***) shall be developed and implemented for all project sites of 1 acre or more to comply with the TCEQ TPDES Construction General Permit TXR1500000. This site-specific current plan must contain the following:

☐ 1. SITE or PROJECT Description that includes:
   a. Description of construction activity and potential pollutants and sources.
   b. Intended schedule or sequence of soil disturbing activities.
   c. Number of acres of the entire construction site property including: off-site material storage areas, overburden and stockpiles of dirt, and borrow areas.
   d. Estimate of runoff coefficient of the site for both pre-construction and post- construction conditions and data describing the soil or the quality of any discharge from the site.
   e. Map showing the general location of the site within the city or county.
   f. Detailed map indicating the following:
      i. Drainage patterns and approximate slopes after grading;
      ii. Areas where soil disturbance will occur;
      iii. Locations of all major structural controls either planned or in place;
      iv. Locations where stabilization practices are expected to be used;
      v. Locations of off-site material, waste, borrow or equipment storage areas;
      vi. surface waters (including wetlands) either adjacent or in close proximity; and,
      vii. Locations where storm water discharges from the site directly to a surface water body.
   g. Location and description of asphalt and concrete plants providing support to construction site under TXR150000.
   h. Name of receiving waters at or near the site that will be disturbed or that will receive discharges from the disturbed area.
   i. A copy of the TPDES general permit.

☐ 2. SWPPP must describe structural and non-structural controls (Best Management Practices) used to minimize pollution from runoff. Must include at a minimum:
   a. Erosion and Sediment Controls
   b. Stabilization Practices

☐ 3. Structural Control Practices to divert flows from exposed soils & limit contact of runoff with disturbed areas to lessen transport of eroded soils.

☐ 4. Permanent Storm Water Controls remaining after construction operations are completed.

☐ 5. Other Controls: off-site tracking of sediments and dust generation, materials storage.

☐ 6. Approved State and Local Plans.

☐ 7. Maintenance of Controls – Must be kept in effective condition.

☐ 8. Inspections of Controls.

☐ 9. Appropriate Controls for Non-storm Water Discharges.

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CHECKLIST C-3

POLLUTION CONTROL PLAN CHECKLIST
CITY OF CORPUS CHRISTI

Development of sites less than 1 acre require a site specific pollution control plan that contain the following:

- ☐ 1. Outline of the site.
- ☐ 2. Delineation of disturbed areas by construction activities.
- ☐ 3. Existing and proposed storm water drainage directional flow lines.
- ☐ 4. Existing and proposed drainage structures.
- ☐ 5. Description of how “run-on” storm water will be handled, including sheet flow entering the site from adjoining property.
- ☐ 6. Description and location of any Environmentally Sensitive Area located on the site or adjoining the site, which will receive storm water directly form the site.
- ☐ 7. Boundary line between any adjoining State submerged land and the site.

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CHECKLIST C-4

EROSION CONTROL PLAN CHECKLIST
CITY OF CORPUS CHRISTI

Please provide complete documentation and details where applicable. NO SWPPP SUBMITTAL will be complete unless all information is filled out completely. Indicate "Not Applicable" where appropriate.

General Information

Name of Development: ______________________________

Address/Location: ______________________________

Plat/Subdivision: ______________________________

Owner Information:

Name: ______________________________

Address: ______________________________

Phone: ______________________________

E-mail address: ______________________________

Developer Information:

Name: ______________________________

Address: ______________________________

Phone: ______________________________

E-mail address: ______________________________

Designer Information:

Name of Company: ______________________________

Address: ______________________________

Engineer Name: ______________________________

Texas Registration Number: ______________________________

Phone: ______________________________

E-mail address: ______________________________

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CHECKLIST C-4

Location Information

Yes	No	N/A	Description	Comments
☐	☐	☐	Project location	
☐	☐	☐	Roads, streets	
☐	☐	☐	North arrow	
☐	☐	☐	Scale	
☐	☐	☐	Property lines	
☐	☐	☐	Existing contours	
☐	☐	☐	Proposed contours	
☐	☐	☐	Limit & acreage of disturbed area	
☐	☐	☐	Planned & existing buildings location & elevations	
☐	☐	☐	Planned & existing roads location & elevations	
☐	☐	☐	Lot and/or building numbers	
☐	☐	☐	Land use of surrounding areas	
☐	☐	☐	Seeps or springs	
☐	☐	☐	Wetland limits	
☐	☐	☐	Easements	
☐	☐	☐	Streams, lakes, ponds, drainage ways, dams	
☐	☐	☐	Stockpiled topsoil or subsoil location	
☐	☐	☐	Street profiles	
☐	☐	☐	Boundaries of the total tract	

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CHECKLIST C-4

Site Drainage Features

Yes	No	N/A	Description	Comments
☐	☐	☐	Existing & planned drainage patterns (include off-site areas that drain through project)	
☐	☐	☐	Size of area (acreage)	
☐	☐	☐	Size & location of culverts & sewers	
☐	☐	☐	Soils information (type, special characteristics)	
☐	☐	☐	Design calculation & construction details for culverts & storm sewers	
☐	☐	☐	Design calculations, cross sections, & method of stabilization of existing & planned channels (include temporary linings)	
☐	☐	☐	Design calculations for peak discharges of runoff (including the construction phase & final runoff coefficients of the site)	
☐	☐	☐	Name of receiving watercourse or municipal operator (only where storm water discharges are to occur)	
☐	☐	☐	Design calculations & construction details of energy dissipaters below culverts & storm sewer outlets (for riprap aprons, include stone sizes & apron dimensions)	
☐	☐	☐	Design calculations & construction details to control groundwater (seeps, high water table, etc.)	

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CHECKLIST C-4

Erosion Control Measures

Yes	No	N/A	Description	Comments
☐	☐	☐	Legend	
☐	☐	☐	Location of temporary and permanent measures	
☐	☐	☐	Construction drawings & details for temporary & permanent measures	
☐	☐	☐	Design calculations for sediment basins & other measures	
☐	☐	☐	Maintenance requirements during construction	
☐	☐	☐	Person responsible for maintenance during construction	
☐	☐	☐	Maintenance requirements & responsible person(s) of permanent measures	

Vegetative Stabilization*

Yes	No	N/A	Description	Comments
☐	☐	☐	Areas & acreage to be vegetatively stabilized	
☐	☐	☐	Planned vegetation with details of plants, seed, mulch, fertilizer	
☐	☐	☐	Specifications for permanent & temporary vegetation	
☐	☐	☐	Method of soil preparation	

*NOTE: Include provisions for ground cover on exposed slopes within 30 working days following completion of any phase of grading; permanent ground cover for all disturbed areas within 30 working days or 120 calendar days (whichever is shorter) following completion of construction or development.

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Watershed Protection

Yes	No	N/A	Description	Comments
☐	☐	☐	Project location	
☐	☐	☐	Watershed classification	
☐	☐	☐	Built upon area (include all existing & proposed buildings & other structures; for non-residential developments include location & size of all built-upon areas including parking & loading facilities)	
☐	☐	☐	Percent of project to be covered with impervious surface	
☐	☐	☐	Proposed number of dwelling units	
☐	☐	☐	Names of adjoining property owners	
☐	☐	☐	Legal description of area storm water control structure (deeded area shall include sufficient area to perform inspection, maintenance, repairs, & reconstruction)	
☐	☐	☐	Impoundment design & calculations	
☐	☐	☐	Maintenance Agreement / Operation & Maintenance Manual	
☐	☐	☐	Performance bond or other financial security for storm water control structure (if required)	

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Other Information

Yes	No	N/A	Description	Comments
☐	☐	☐	Completed Financial Responsibility/ Ownership statement (signed by person financially responsible for project)	
☐	☐	☐	Construction sequence related to sedimentation & erosion control (include installation of critical measures prior to initiation of land- disturbing activity & removal of measures after permanent stabilization)	
☐	☐	☐	State on plan if there *is* or *is not* a floodplain associated with project, and provide elevation & location	
☐	☐	☐	If the project is affected by any watershed protection ordinance, supply a short letter describing the watershed protection method being used	
☐	☐	☐	Narrative describing the nature & purpose of the construction activity	

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CHECKLIST C-5

SAMPLE WEEKLY SITE INSPECTION CHECKLIST (FOR SITE MANAGERS)

The purpose of this inspection checklist is to provide the Site Manager with a list of storm water and erosion control devices/measures that must be inspected weekly. The Site Manager should correct damage or deficiencies, and any changes that may be required to correct deficiencies in the SWPPP should be made as soon as possible after the inspection.

Project ______________________________

Location ______________________________

Inspector ______________________________

Date ______________________ Time ______________________

Indicate which types of devices/measures are used on the site (check all that apply).

Stabilization Practices:

- ☐ Seeding
- ☐ Mulching
- ☐ Sod stabilization
- ☐ Vegetative buffer strips
- ☐ Protection of Trees

Structural Practices – Runoff Conveyance:

- ☐ Permanent drainways
- ☐ Sodding
- ☐ Grassed waterways Reinforced
- ☐ grassed waterways Ripraps
- ☐ Lined waterways
- ☐ Inlet protection barriers
- ☐ Inlet insert baskets
- ☐ Other (please describe below):

____

Structural Practices – Erosion Control:

- ☐ Temporary diversion dikes & channels
- ☐ Downspouts & discharge outlets

Structural Practices – Sediment Control:

- ☐ Filter fabric fences & barriers
- ☐ Straw bale fences
- ☐ Sediment traps Sediment
- ☐ basins Vegetative buffer
- ☐ strips Sand bags
- ☐ Sediment tanks
- ☐ Sediment sump pits
- ☐ Temporary stabilized access roads & parking areas
- ☐ Street sweeping & vacuuming
- ☐ Hosing tires & treads

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CHECKLIST C-5

Indicate the condition of each type of device/measure that was checked on page C-5:1. Also indicate if any maintenance is required, and when maintenance shall be completed. Attach additional sheets as needed.

1. Type of Device/Measure: ______________________________

a. Condition: ______________________________

____

____

b. Overall condition of device/measure (check one):
_______ Acceptable
_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

2. Type of Device/Measure: ______________________________

a. Condition: ______________________________

b. Overall condition of device/measure (check one):
_______ Acceptable
_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

3. Type of Device/Measure: ______________________________

a. Condition: ______________________________

b. Overall condition of device/measure (check one):
_______ Acceptable
_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

____

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____
Inspector's Signature

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CHECKLIST C-6

SAMPLE POST CONSTRUCTION INSPECTION CHECKLIST (FOR CITY INSPECTORS)

(See Section 9.5)

The purpose of this inspection checklist is to provide the City Inspector with a list of storm water and erosion control devices/measures that must be inspected after construction is complete, to ensure that BMPs listed in the Storm Water Quality Management Plan have been implemented. All items must be satisfied before an occupancy permit will be issued.

Project ____

Location ____

Inspector ____

Date ____________________ Time ____________________

Indicate which types of devices/measures are used on the site (check all that apply).

Nonstructural Practices:

- ☐ Inlet Stenciling

Permanent Stabilization Practices:

- ☐ Sodding
- ☐ Seeding
- ☐ Mulching

Structural Practices – Runoff Conveyance:

- ☐ Dikes & Swales
- ☐ Backslope Drain
- ☐ Paved Flume
- ☐ Level Spreader

Structural Practices – Sediment Control:

- ☐ Rock rubble berm
- ☐ Stone outlet sediment trap
- ☐ Excavated earth outlet sediment trap

Structural Practices – Erosion Control:

- ☐ Grass-lined channels
- ☐ Concrete Slope Paving
- ☐ Interlocking Blocks
- ☐ Riprap

Structural Practices – Energy Dissipation:

- ☐ Riprap Apron Riprap
- ☐ Stilling Basin
- ☐ USBR Type VI Impact Basin St.
- ☐ Anthony Falls stilling basin
- ☐ Other ____________________

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CHECKLIST C-6

Structural Best Management Practices:

- ☐ Extended dry detention basins
- ☐ Retention (Wet) Ponds
- ☐ Constructed Wetlands
- ☐ Grassed Swales
- ☐ Filter Strips and Flow Spreaders
- ☐ Sand Filters
- ☐ Infiltration Trenches
- ☐ Porous Pavement
- ☐ Oil/Grit Separators
- ☐ Catch Basin Inserts
- ☐ Litter Traps

☐ **Other (please describe below):**

____

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CHECKLIST C-6

Indicate the condition of each type of device/measure that was checked on pages C-6:1 through C-6:2. Also indicate if any maintenance is required, and when maintenance shall be completed. Attach additional sheets as needed.

1. Type of Device/Measure: ______________________________

a. Condition: ______________________________
____
____

b. Overall condition of device/measure (check one):
_______ Acceptable
_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

2. Type of Device/Measure: ______________________________

a. Condition: ______________________________
____
____
____

b. Overall condition of device/measure (check one):
_______ Acceptable
_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________
____
____
____

3. Type of Device/Measure: ______________________________

a. Condition: ______________________________
____
____
____

b. Overall condition of device/measure (check one):
_______ Acceptable
_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________
____
____

____
Inspector's Signature

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4. Type of Device/Measure: ______________________________

a. Condition: ______________________________

____

____

____

b. Overall condition of device/measure (check one):

_______ Acceptable

_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

____

____

5. Type of Device/Measure: ______________________________

a. Condition: ______________________________

____

____

____

b. Overall condition of device/measure (check one):

_______ Acceptable

_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

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____

6. Type of Device/Measure: _

a. Condition: ______________________________

____

____

____

b. Overall condition of device/measure (check one):

_______ Acceptable

_______ Unacceptable

c. Dates any maintenance must be completed by: ______________________________

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Inspector's Signature

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CHECKLIST C-7

FACILITY MAINTENANCE CHECKLIST
CITY OF CORPUS CHRISTI

The purpose of the enclosed inspection sheets is to provide a maintenance plan to ensure the continued proper operation of all storm water facilities. Lack of maintenance could lead to local flooding, water damage and costly repairs or replacements of these or other infrastructure.

Contents:

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CHECKLIST C-7

ACCESS ROADS & EASEMENTS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Project____________________

Location ____________________

Inspector ____________________

Date ____________________ Time ____________________

Inspection Frequency Key: A=Annual, M=Monthly, Q=Quarterly, S=After Major Storm

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
Access/Easement Components				
1. Debris Removal				
a. Is surface clear of debris and trash?			Q	
b. Any obstructions restricting the access road surface to less than 15'?			Q	
2. Vegetation				
a. Are weeds that are more than 6" tall growing in the road surface?			M	
3. Erosion				
a. Any settlement, potholes, soft spots, or ruts that exceed 6" in depth and 6 ft^2 in area in the road surface?			A	
b. Any erosion within 1 ft of roadway more than 8" wide & 6" deep?			A	

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CHECKLIST C-7

ACCESS ROADS & EASEMENTS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Summary

a. Inspector's remarks: ____

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____
____
____
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____
____

b. Overall condition of facility (check one):

_______ Acceptable

_______ Unacceptable

c. Dates any maintenance must be completed by:

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Inspector's Signature

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CHECKLIST C-7

CATCH BASINS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Project ____

Location ____

Inspector ____

Date ____________________ Time ____________________

Inspection Frequency Key: A=Annual, M=Monthly, Q=Quarterly, S=After Major Storm

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
Access/Easement Components				
1. Debris Removal				
a. Is basin clear of debris and trash?			A,S	
b. Is Sediment buildup 6” or greater (if yes, removal required)?			A,S	
2. Structure				
a. Is ladder unsafe due to missing rungs, misalignment, rust, or cracks?			A	
b. Condition of access cover?			A	

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CHECKLIST C-7

CATCH BASINS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Summary

1. Inspector's remarks: ____

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2. Overall condition of facility (check one):

_______ Acceptable

_______ Unacceptable

3. Dates any maintenance must be completed by:

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Inspector's Signature

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CHECKLIST C-7

OPEN CHANNEL
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Project ____

Location ____

Inspector ____

Dat ______________________ Time ______________________

Inspection Frequency Key: A=Annual, M=Monthly, Q=Quarterly, S=After Major

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
Channel Components				
1. Debris Cleanout				
a. Contributing areas clean of debris?			M	
2. Check Dams or Energy Dissipators				
a. Any evidence of flow going around structures?			A,S	
b. Any evidence of erosion at downstream toe?			A,S	
c. Soil Permeability.			A,S	
d. Groundwater/bedrock.			A,S	
3. Vegetation				
a. Mowing done when needed?			M	
b. Minimum mowing depth not exceeded?			M	
c. Any evidence of erosion?			M	
d. Fertilized per specification?			M	
e. Undesirable vegetative growth?			M	
f. Undesirable woody vegetation?			M	
4. Dewatering				
a. Dewaters between storms?			M	
5. Sediment Deposition				
a. Clean of sediment?			A	
6. Outlet/Overflow Spillway				
a. Condition of spillway?			A	
b. Any evidence of erosion?			A	

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CHECKLIST C-7

OPEN CHANNEL MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Summary

1. Inspector's remarks: ____

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____

2. Overall condition of facility (check one):

   _______ Acceptable

   _______ Unacceptable

3. Dates any maintenance must be completed by:

____
____
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____

____

Inspector's Signature

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CHECKLIST C-7

SAND/ORGANIC FILTRATION FACILITY
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Project ______________________________

Location ______________________________

Inspector ______________________________

Date ________________ Time ________________

Inspection Frequency Key: A=Annual, M=Monthly, Q=Quarterly, S=After Major Storm

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
Facility Components				
1. Debris Cleanout				
a. Contributing areas clean of debris?			M	
b. Inlets and outlets clear of debris?			M	
c. Filtration facility clean of debris?			M	
2. Vegetation				
a. Contributing drainage area stabilized?			M	
b. Grass mowed & clippings removed?			M	
c. Any evidence of erosion?			M	
3. Oil and Grease				
a. Any evidence of filter surface clogging?			M	
b. Do activities in drainage area minimize oil & grease entry?			M	
4. Water Retention Where Required				
a. Water holding chambers at normal pool?			M	
b. Any evidence of leakage?			M	
5. Sediment Deposition				
a. Filtration chamber free of sediments?			A	
b. Sedimentation chamber not more than half full of sediments?			A	

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CHECKLIST C-7

SAND/ORGANIC FILTRATION FACILITY
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
6. Structural components				
a. Any evidence of structural deterioration?			A	
b. Condition of grates?			A	
c. Any evidence of spalling or cracking of structural parts?			A	
7. Outlets/Overflow Spillway				
a. Condition of outlet structures?			A	
b. Any evidence of erosion?			A	
8. Overall Function of Facility				
a. Any evidence of flow bypassing facility?			A	
b. Any noticeable odors outside of facility?			A	
9. Pump (Where Applicable)				
a. Are catalog cuts and wiring diagram for pump available?			A	
b. Do waterproof conduits for wiring appear to be intact?			A	
c. Panel box well marked?			A	
d. Any evidence of pump failure (excess water in pump well, etc.)?			A	

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CHECKLIST C-7

SAND/ORGANIC FILTRATION FACILITY
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Summary

1. Inspector's remarks: ____

2. Overall condition of facility (check one):
   _______ Acceptable

   _______ Unacceptable

3. Dates any maintenance must be completed by:

____
Inspector's Signature

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CHECKLIST C-7

STORM WATER POND/WETLAND
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Project____

Location____

Inspector ____

Date ____________________ Time ______________________

Inspection Frequency Key: A=Annual, M=Monthly, Q=Quarterly, S=After Major Storm

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
Pond Components				
1. Embankment & Emergency Spillway				
a. Adequate vegetation & ground cover?			A	
b. Embankment erosion?			A	
c. Animal burrows?			A	
d. Unauthorized plantings?			A	
e. Cracking, bulging, or sliding of berm/dam?				
i. Upstream face			A	
ii. Downstream face			A	
iii. At or beyond toe				
Upstream			A	
Downstream			A	
iv. Emergency spillway			A	
f. Pond, toe & chimney drains clear & functioning?			A	
g. Leaks on downstream face?			A	
h. Slope protection or riprap failures?			A	
f. Visual, settlement or horizontal misalignment of top of dam?			A	
j. Emergency spillway clear of debris?			A	
k. Other (specify)?			A	

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CHECKLIST C-7

STORM WATER POND/WETLAND MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
2. Spillway				
a. Low flow orifice obstructed?			A	
b. Low flow trash rack (if applicable)				
i. Debris removal necessary?			A	
ii. Corrosion control needed?			A	
c. Weir trash rack (if applicable)				
i. Debris removal necessary?			A	
ii. Corrosion control needed?			A	
d. Excessive sediment accumulation inside riser?			A	
e. Concrete/masonry condition riser & barrels				
i. Cracks or displacements ?			A	
ii. Minor spalling (<1”)?			A	
iii. Major spalling (rebars exposed)?			A	
iv. Joint failures?			A	
v. Water tightness?			A	
f. Metal pipe condition?			A	
g. Control Valve				
i. Operational/exercised?			A	
ii. Chained and locked?			A	
h. Pond drain valve				
i. Operational/exercised?			A	
ii. Chained and locked?			A	
i. Outfall channels flowing?			A	
j. Other (specify)?			A	
3. Permanent Pool (Wet Ponds)				
a. Undesirable vegetative growth?			M	
b. Floating or floatable debris present?			M	
c. Visible pollution?			M	
d. High Water Marks?			M	
e. Shoreline problems?			M	
f. Other (specify)?			M	

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CHECKLIST C-7

STORM WATER POND/WETLAND
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
4. Sediment Forebays				
a. Sedimentation noted?			M	
b. Sediment removal when depth <50% design depth.			M	
5. Dry Pond Areas				
a. Vegetation adequate?			M	
b. Undesirable vegetative growth?			M	
c. Undesirable woody vegetation?			M	
d. Low flow channels clear of obstructions?			M	
e. Standing water or wet spots?			M	
f. Sediment and/or trash accumulation?			M	
g. Other (specify)?			M	
6. Condition of Outfalls into Pond				
a. Riprap failures?			A,S	
b. Slope erosion?			A,S	
c. Storm drain pipe(s) condition?			A,S	
d. Endwalls/headwalls condition?			A,S	
e. Other (specify)?			A,S	
7. Other				
a. Encroachments on ponds or easement area?			M	
b. Complaints from residents (describe)?			M	
c. Disease carrying animals/insects?			M	
d. Aesthetics				
i. Grass height.			M	
ii. Graffiti removal necessary?			M	
iii. Other (specify)?			M	
e. Public hazards (specify)?			M	
f. Maintenance access unimpaired?			M	
8. Constructed Wetland Areas				
a. Vegetation healthy and growing?			A	
b. Evidence of invasive species?			A	
c. Excessive sedimentation in wetland area?			A	

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CHECKLIST C-7

STORM WATER POND/WETLAND MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Summary

1. Inspector's remarks: ______________________________

2. Overall condition of facility (check one):
   _______ Acceptable

   _______ Unacceptable

3. Dates any maintenance must be completed by:

____
Inspector's Signature

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CHECKLIST C-7

SWALES, GRASS CHANNELS, & FILTER STRIPS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Project____

Location____

Inspector ____

Date ______________________ Time ______________________

Inspection Frequency Key: A=Annual, M=Monthly, Q=Quarterly, S=After Major Storm

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
Components				
1. Debris Removal				
a. Facility & adjacent area clear of debris?			M	
b. Inlets & outlets clear of debris?			M	
c. Any dumping of yard wastes into facility?			M	
d. Has litter (branches, etc.) been removed?			M	
2. Vegetation				
a. Adjacent area stabilized.			M	
b. Grass mowed.			M	
c. Plant height should not be less than design water depth.			M	
d. Fertilized per specifications?			M	
e. Any evidence of erosion?			M	
f. Is plant composition according to approved plans?			M	
g. Any unauthorized or inappropriate plantings?			M	
h. Any dead or diseased plants?			M	
i. Any evidence of plant stress from inadequate watering?			M	
j. Any evidence of deficient stakes or wires?			M	

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CHECKLIST C-7

SWALES, GRASS CHANNELS, & FILTER STRIPS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Inspection Items	Checked? Yes/No	Maintenance Needed? Yes/No	Inspection Frequency	Comments
3. Oil and Grease				
a. Any evidence of filter clogging?			M	
4. Dewatering				
a. Facility dewaters between storms?			M	
5. Check Dams/Energy Dissipators/Sumps				
a. Any evidence of sedimentation buildup?			A,S	
b. Are sumps greater than 50% full of sediment?			A,S	
c. Any evidence of erosion at downstream toe of drop structures?			A,S	
6. Sediment Deposition				
a. Swale clean of sediments?			A	
b. Sediments should not be greater than 20% of swale design depth.			A	
7. Outlets/Overflow Spillway				
a. Condition of outlet structures.			A,S	
b. Any evidence of erosion?			A,S	
c. Any evidence of blockages?			A,S	
8. Integrity of Facility				
a. Has facility been blocked or filled inappropriately?			A	
9. Bioretention Planting Soil				
a. Any evidence of planting soil erosion?			A	
10. Organic Layer				
a. Mulch covers entire area (NO voids) and to specified thickness.			A	
b. Mulch is in good condition.			A	

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CHECKLIST C-7

SWALES, GRASS CHANNELS, & FILTER STRIPS
MAINTENANCE AND MANAGEMENT INSPECTION CHECKLIST

Summary

1. Inspector's remarks: ____

____

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____

____

2. Overall condition of facility (check one):
   _______ Acceptable

   _______ Unacceptable

3. Dates any maintenance must be completed by:

____

____

____

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____

Inspector's Signature

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CHECKLIST C-8

STORM WATER PLAN SUBMITTALS
CITY OF CORPUS CHRISTI

The types of engineering submittals made in connection with new development or drainage studies include the following:

1. **Engineering Reports:** These documents, in letter report or formal bound report form, describe the results of analyses of existing and/or proposed drainage conditions. Engineering reports may be submitted as a basis for:
   - better understanding of existing conditions (e.g., a flood plain revision report),
   - supporting a request for approval of construction documents for a proposed facility (e.g., a preliminary engineering report for a roadway improvement project),
   - or, serving as a plan for future conditions (e.g., a master drainage report for a given drainage Basin).
2. **Construction Documents:** These include engineering drawings and specifications for a proposed facility or development which will affect storm water drainage or flood protection.
3. **Permit Applications:** City permit requirements for project development include building permits, engineering permits, flood plain fill permits, excavation permits, and other applicable regulatory permits.

The City of Corpus Christi requires that engineering submittals be prepared for all activities which may affect the rate, direction, or volume of storm runoff, or the depth and velocity of flow in drainage systems within the City's incorporated boundaries or extraterritorial jurisdiction. Associated with each of these activities is the potential for adverse impacts. It is for this reason that the City of Corpus Christi regulates these types of activities, which include, but are not limited to, the following:

- New development/redevelopment
- Detention design
- Channel improvements
- New channel structures (bridges, culverts, etc.)
- Flood plain reclamation (fill)
- Hydraulic studies
- Hydrologic studies
- Minor drainage improvements

All reports should include all of the necessary information, utilizing text, tables, and exhibits to thoroughly document the methods, data, and assumptions used in completing analyses of the proposed activity as well as the results obtained. Detailed computation results shall be attached to the report. All maps and other exhibits must be legible and information should be presented clearly and concisely. Checklist C-8 should be included with all submittals.

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CHECKLIST C-8

STORM WATER PLAN SUBMITTALS
CITY OF CORPUS CHRISTI

Please provide complete documentation and details where applicable. NO PLAN SUBMTTAL will be complete unless all information is filled out completely. Indicate "Not Applicable" where appropriate.

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General Information

Name of Development: ______________________________

Address/Location: ______________________________

Plat/Subdivision: ______________________________

Owner Information:

Name: ______________________________

Address: ______________________________

Phone: ______________________________

E-mail address: ______________________________

Developer Information:

Name: ______________________________

Address: ______________________________

Phone: ______________________________

E-mail address: ______________________________

Engineer of Record Information:

Name of Company: ______________________________

Firm Registration Number: : ______________________________

Address: ______________________________

____

Engineer's Name: ______________________________

Texas Registration Number: ______________________________

Phone: ______________________________

E-mail Address: ______________________________

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Plan Requirements:

Yes	No	N/A	Description	Remarks
☐	☐	☐	Development Name	
☐	☐	☐	Owner	
☐	☐	☐	Engineer of Record	
☐	☐	☐	Plan Set (circle one) Preliminary, Revised, Final	
☐	☐	☐	Final Plan Set Sealed, Signed and Dated	
☐	☐	☐	Plan Set Sheets	
☐	☐	☐	Date	
☐	☐	☐	North Arrow	
☐	☐	☐	Property Lines	
☐	☐	☐	Legend	
☐	☐	☐	Vicinity Map	
☐	☐	☐	Site Map - Includes Environmentally Sensitive Areas Analysis (circle one) Yes, No	
☐	☐	☐	Drainage Map	
☐	☐	☐	Flood Plain Map	
☐	☐	☐	Facility Layout Map	
☐	☐	☐	Scale	
☐	☐	☐	Adjacent Property Owners	
☐	☐	☐	Existing Streets, Buildings, etc.	
☐	☐	☐	Wetland Limits	
☐	☐	☐	Easements	
☐	☐	☐	Right-of-Way Map	
☐	☐	☐	Land Use of Surrounding Areas	
☐	☐	☐	Original Contours (Typ. 1’ Intervals))	
☐	☐	☐	Field Survey Data	
☐	☐	☐	LiDAR Data	

Plan Requirements (continued):

Yes	No	N/A	Description	Remarks
☐	☐	☐	Benchmark Information	
☐	☐	☐	Existing Streams, Lakes, etc.	
☐	☐	☐	Proposed Contours (2-foot intervals)	
☐	☐	☐	Size & Location of Existing Culverts	
☐	☐	☐	Size & Location of Proposed Culverts	
☐	☐	☐	Datum Elevation	
☐	☐	☐	Plotted Stream Profile	
☐	☐	☐	Plotted Cross-Sections	
☐	☐	☐	Storm Water Management Plan (Drainage Report)	
☐	☐	☐	Storm Water Quality Management Plan	
☐	☐	☐	Storm Water Pollution Prevention Plan	
☐	☐	☐	Pollution Control Plan	

Calculation Requirements:

NOTE: Calculations for peak flows, pipe/channel/street capacity and velocity calculations, outfall velocity, culverts/bridges, detention and HGL/EGL must be on plan sheets and in the drainage report.

Yes	No	N/A	Description	Remarks
☐	☐	☐	Design Assumptions ______	
☐	☐	☐	Design Calculations ______ Discharge Calculations ______ Hydraulic Calculations Hydrologic Impacts Analysis Hydraulic Impacts Analysis _______________	
☐	☐	☐	Drainage Easement/ROW	

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CHECKLIST C-8

Closed Systems:

Yes	No	N/A	Description	Remarks
☐	☐	☐	Closed System Type (circle one) Minor, Collector, Major	
☐	☐	☐	Designed for 5-yr Storm	
☐	☐	☐	Designed for 25-yr Storm	
☐	☐	☐	Designed for 100-yr Storm	
☐	☐	☐	Analyzed for 25-yr Storm	
☐	☐	☐	Analyzed for 100-yr Storm	
☐	☐	☐	100-yr Extreme Event Conveyance	
☐	☐	☐	Minimum/maximum velocity	
☐	☐	☐	Minimum Cover = 2 Feet	
☐	☐	☐	Outfall Details/Calculations	
☐	☐	☐	Energy Dissipator Calculations	
☐	☐	☐	Evaluation of Downstream System	
☐	☐	☐	Catch Basin Designed for 5-yr Storm	
☐	☐	☐	Drainage Easement/ROW	
☐	☐	☐	Recommendations for Adverse Impacts Mitigation	

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CHECKLIST C-8

<u>Open Channel Systems</u>:

Yes	No	N/A	Description	Remarks
☐	☐	☐	Open Channel Type (circle one) Collector, Major	
☐	☐	☐	Designed for 25-yr Storm	
☐	☐	☐	Designed for 100-yr Storm	
☐	☐	☐	Analyzed for 100-yr Storm	
☐	☐	☐	Side slopes 4 to 1 or Flatter	
☐	☐	☐	Minimum Bottom Width = 6 Feet for Earthen 8 Feet for Concrete-Lined	
☐	☐	☐	Velocity Check (Liners Provided, If Needed)	
☐	☐	☐	Evaluation of Downstream System	
☐	☐	☐	Evaluation of Upstream System	
☐	☐	☐	Drainage Easement/ROW	
☐	☐	☐	Recommendations for Adverse Impacts Mitigation	

Yes	No	N/A	Description	Remarks
☐	☐	☐	Crossing Structure Type (circle one) Minor, Collector, Major	
☐	☐	☐	Designed for 5-yr Storm	
☐	☐	☐	Designed for 25-yr Storm	
☐	☐	☐	Designed for 100-yr Storm	
☐	☐	☐	Analyzed for 25-yr Storm	
☐	☐	☐	Analyzed for 100-yr Storm	
☐	☐	☐	Maximum Velocity	
☐	☐	☐	Erosion Protection Upstream & Downstream for Earthen Channels	
☐	☐	☐	Energy Dissipator Calculations	
☐	☐	☐	Drainage Easement/ROW	
☐	☐	☐	Recommendations for Adverse Impacts Mitigation	

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CHECKLIST C-8

Detention Facilities:

Yes	No	N/A	Description	Remarks
☐	☐	☐	Analyzed for 2-yr Storm	
☐	☐	☐	Analyzed for 10-yr Storm	
☐	☐	☐	Analyzed for 100-yr Storm	
☐	☐	☐	Maximum side slopes	
☐	☐	☐	Minimum bottom slope	
☐	☐	☐	Minimum Freeboard Provided = 1.0 ft	
☐	☐	☐	Pilot Channel Provided	
☐	☐	☐	Outfall Pipe (18” minimum)	
☐	☐	☐	Emergency Spillway	
☐	☐	☐	Maintenance Strip/ROW	
☐	☐	☐	Multi-Purpose Design	
☐	☐	☐	Recommendations for Adverse Impacts Mitigation	

Environmentally Sensitive Areas:

Yes	No	N/A	Description	Remarks
☐	☐	☐	Environmentally Sensitive Areas Identified	
☐	☐	☐	Vegetated Buffer Strips Provided	
☐	☐	☐	Velocity Control at Outfall Provided	
☐	☐	☐	Sediment Reduction BMPs Provided	
☐	☐	☐	Floatables Collection BMPs Provided	
☐	☐	☐	Oil/Gas Removal BMPs Provided	

Other Plan Submittals:

Yes	No	N/A	Description	Remarks
☐	☐	☐	TxDOT	
☐	☐	☐	US Army Corps of Engineers	
☐	☐	☐	TCEQ	

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APPENDIX D

REFERENCES

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