2011

Storm Drainage Design Manual



City of Gillette
Engineering Department
201 E. 5th Street • Gillette, WY 82716 • 303.686.5265

URS

CITY OF GILLETTE

STORM DRAINAGE DESIGN MANUAL

DEPARTMENT OF ENGINEERING

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LIST OF ACRONYMS

AASHTO	America Association of State Highway and Transportation Officials
ACPA	American Concrete Pipe Association
A.D.T.	average daily traffic
AFOs	Animal Feeding Operations
AISI	American Iron and Steel Institute
ASCE	American Society of Civil Engineers
BMP	Best Management Practice
CAP	Corrugated Aluminum Pipe
CDOT	Colorado Department of Transportation
cfs	cubic feet per second
CLOMR	Conditional Letter of Map Revision
CMP	Corrugated Metal Pipe
EDB	extended detention basins
EGL	Energy Grade Line
FAA	Federal Aviation Administration
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
fps	feet per second
ft	foot/feet
GSB	grouted sloping boulder
HGL	Hydraulic Grade Line
HSG	Hydrologic Soil Group
IBC	International Building Code
in	inches
LID	Low Impact Development
MS4	Municipal Separate Storm Sewer System
NOAA	National Oceanic and Atmospheric Administration
NOT	Notice of Termination
NOTA	Notice of Transfer and Acceptance
NRCS	Natural Resources Conservation Service

LIST OF ACRONYMS

OSHA	Occupational Safety and Health Administration
RCP	Reinforced Concrete Pipe
ROW	Right-of-Way
RP	retention pond
SEO	State Engineer’s Office
SWMM	Stormwater Management Model
SWMP	Stormwater Management Plan
UDFCD	Urban Drainage and Flood Control District
UH	Unit Hydrograph
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USDCM	Urban Storm Drainage Criteria Manual
USDOT	United States Department of Transportation
VHB	vertical hard basin
WQCV	Water Quality Capture Volume
WYDEQ	Wyoming Department of Environmental Quality
WYDOT	Wyoming Department of Transportation

DEFINITIONS

The following terms and parameters are defined and must be understood when analyzing open-channel flows:

Area (A): The area is the cross-sectional area of the flow, measured perpendicular to the direction of flow.

Wetted Perimeter (P): The wetted perimeter is the length along the bottom and edges of the cross-section of the flow conveyance facility that is in contact with flowing water.

Hydraulic Radius (R): The hydraulic radius is the cross-sectional area of flow divided by the wetted perimeter, or $R = A/P$.

Depth (d): If not specified otherwise, depth of flow refers to the maximum depth of water in the cross-section.

Surface Spread (T): The surface spread is the width at the top of the flow, measured perpendicular to the flow direction.

Hydraulic Depth (Dh): The hydraulic depth is the ratio of area of flow to the width of the channel at the water surface, or $Dh = A/T$.

Slope (S): Slope may refer to the channel bed, the hydraulic grade line, or energy grade line, expressed in ft per ft.

Hydraulic Grade Line (HGL): In an open channel, the hydraulic grade line is the profile of the free water surface.

Hydraulic Gradient (Hg): The Hydraulic Gradient is the slope of the hydraulic grade line in the profile of the free water surface.

Hydrophytic Vegetation: The sum total of macrophytic plant life that occurs in areas where the frequency and duration of inundation or soil saturation produce permanently or periodically saturated soils of sufficient duration to exert a controlling influence on the plant species present" (U.S. Army Corps of Engineers 1987).

Energy Grade Line (EGL): The Energy Grade Line is the grade line of the water surface profile plus the velocity head, or the specific energy line.

Critical Flow: This is flow at critical depth or velocity, where the specific energy is a minimum for a given discharge. Critical flow is very unstable.

Critical Depth: (Dc): This is the depth of flow under critical flow conditions.

Critical Velocity (Vc): This is the velocity of flow under critical flow conditions.

Critical Slope: This refers to the slope of the conveyance facility which, for a given cross-section and flow rate, results in critical flow.

Froude Number (Fr): This dimensionless number is a measure of whether flow is subcritical, critical, or supercritical.

Normal Depth: In a uniform conveyance facility on a constant slope, normal depth is the flow depth along a channel reach, when neither the flow depth nor velocity is changing.

DEFINITIONS

Open-Channel Flow: This is any water flow that is conveyed such that top surface is exposed to the atmosphere.

Uniform Flow: Uniform flow occurs when flow has a constant water area, depth, discharge, and average velocity through a conveyance facility.

Gradually Varied Flow: This is flow in which the depth does not change abruptly over a comparatively short distance.

Thalweg: This is the line defining the lowest elevations along the length of a river bed or valley.

Finally Stabilized: This means that all soil disturbing activities at the site have been completed, and a uniform perennial vegetative cover with a density of 70% of the native background vegetative cover for the area has been established, or another form of ground cover that protects the soil from erosion has been installed, on all disturbed unpaved areas and areas not covered by permanent structures.

ACKNOWLEDGEMENTS

The City acknowledges and thanks the following individuals and organizations for their contributions to this manual.

TOM MURPHY, MAYOR

CITY COUNCIL

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Louise Carter-King	Ted Jerred
Robin Kuntz	John Opseth

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This manual relies significantly on work previously completed by others in several existing storm drainage manuals including:

Urban Storm Drainage Criteria Manual, Volumes 1 – 3 (UDFCD 2008)

Mesa County, City of Grand Junction Stormwater Management Manual (December 2007)

The City of Casper Erosion and Sediment Control Best Management Practices Manual

Town of Castle Rock Storm Drainage Design and Technical Criteria Manual (2005)

The City of Rapid City Stormwater Quality Manual (2009)

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ADOPTING ORDINANCE

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ORDINANCE NO. 3780

AN ORDINANCE AMENDING SECTIONS 7-1 AND 7-2 OF THE GILLETTE CITY CODE, TO ADOPT THE CITY OF GILLETTE 2012 STANDARD CONSTRUCTION SPECIFICATIONS AND THE CITY OF GILLETTE 2012 DESIGN STANDARDS, RENUMBERING SECTION 7-5 TO 7-6 AND AMENDING SECTION 7-5 TO ADOPT THE 2011 STORM DRAINAGE DESIGN MANUAL AND SETTING AN EFFECTIVE DATE OF JANUARY 1, 2013

BE IT ORDAINED BY THE GOVERNING BODY OF THE CITY OF GILLETTE, WYOMING:

SECTION ONE. Section 7-1 of the Gillette City Code is amended to read as follows:

§ 7-1. ADOPTION OF CITY OF GILLETTE 2012 STANDARD CONSTRUCTION SPECIFICATIONS

The City of Gillette 2012 Standard Construction Specifications, prepared by the Department of Engineering of the City of Gillette is hereby adopted by this reference and incorporated herein as if set out in full to regulate construction within the City of Gillette starting January 1, 2013. One copy shall be available for public inspection at the office of the Department of Engineering and the office of the City Clerk during normal business hours. The Department of Engineering will also provide copies for sale at a reasonable charge to cover the cost of preparation of the volume.

SECTION TWO. Section 7-2 of the Gillette City Code is enacted to read as follows:

§ 7-2. ADOPTION OF CITY OF GILLETTE 2012 DESIGN STANDARDS

The City of Gillette 2012 Design Standards, prepared by the Department of Engineering of the City of Gillette is hereby adopted by this reference and incorporated herein as if set out in full to regulate the design of public improvements within the City of Gillette starting January 1, 2013. One copy shall be available for public inspection at the office of the Department of Engineering and the office of the City Clerk during normal business hours. The Department of Engineering will also provide copies for sale at a reasonable charge to cover the cost of preparation of the volume.

SECTION THREE. Section 7-5 of the Gillette City Code is renumbered Section 7-6, and Section 7-5 is amended to read as follows:

§ 7-5 ADOPTION OF THE 2011 STORM DRAINAGE DESIGN MANUAL

The City of Gillette 2011 Storm Drainage Design Manual, prepared by the Department of Engineering of the City of Gillette is hereby adopted by this reference and incorporated herein as if set out in full to regulate the design of public improvements within the City of Gillette starting January 1, 2012. One copy shall be available for public inspection at the office of the

Department of Engineering and the office of the City Clerk during normal business hours. The Department of Engineering will also provide copies for sale at a reasonable charge to cover the cost of preparation of the volume.

SECTION FOUR. This ordinance shall be effective on January 1, 2013.

PASSED, APPROVED AND ADOPTED this 17th day of December, 2012.

Tom Murphy, Mayor

(SEAL)
ATTEST:

Karlene Abelseth

Karlene Abelseth, City Clerk

Published: December 26, 2012

INTRODUCTION

The City of Gillette Storm Drainage Design Manual contains design standards to be used for public and private development and re-development projects and for City contracted projects. The Storm Drainage Design Manual also contains general pre-construction requirements and construction requirements for stormwater permitting and obtaining approval of Stormwater Pollution Prevention Plans for development projects and other public improvements.

This Storm Drainage Design Manual is intended to cover the drainage design methods and to give the minimum values the City will accept. The City Engineer encourages the use of the design methods given, but will accept proven alternative design methods.

When submitting the plans and reports for approval by the City Engineer, calculations should be included. The calculations should be submitted in a neat and readable fashion but need not be typed.

Where a reference is made to any ASTM, ANSI, AASHTO, DEQ, MUTCD, or any other standardized document or designation, it shall be the latest revision at the time.

RELATIONSHIP TO PREVIOUS VERSIONS OF GILLETTE STORM DRAINAGE CRITERIA MANUAL

This *Gillette Storm Drainage Design Manual* updates and supersedes the previous *Gillette Storm Drainage Criteria*, as published in the 2009 Design Standards, Section 403. A few noteworthy changes include:

- New inlet and street capacity charts.
- New requirements and details for detention and water quality facilities.
- New emphasis on stormwater quality and construction requirements to reflect City of Gillette Stormwater Discharge Permit requirements with extensive cross-referencing to the Urban Drainage and Flood Control District (UDFCD) *Urban Storm Drainage Criteria Manual*, as updated in 2008 and as may be periodically amended.
- New drainage and construction plan submittal checklists to improve user friendliness for developers and their engineers.

Where conflicts regarding drainage design elements for public improvements between this Storm Drainage Design Manual and the City Design Standards exist, this Storm Drainage Design Manual shall govern.

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SECTION ONE
GENERAL

1.1 PURPOSE AND SCOPE

This manual, together with all future changes and amendments, shall be known as the *City of Gillette Storm Drainage Design Manual* (hereinafter called Drainage Criteria) as referenced in the *City of Gillette Stormwater Ordinance.* The purpose of these Drainage Criteria is to provide requirements and guidance necessary for developers, consultants, and industrial and commercial operators to select, design and maintain drainage and flood control facilities. The ultimate goal of these Drainage Criteria is to protect the public health, safety and welfare and minimize adverse impacts to the environment.

These Drainage Criteria shall apply to all land within the incorporated areas of the City of Gillette (City), including any public lands, and all major subdivisions and developments, site plan and residential building permit applications, as well as existing residential, commercial, industrial, and institutional properties, and other properties outside the City of Gillette where the City of Gillette has subdivision approval authority unless eligible for an exemption or granted a waiver by the City of Gillette. These Drainage Criteria shall apply to all facilities constructed in City ROW, easements dedicated for public use, and to all privately owned and maintained drainage facilities, including, but not limited to, detention facilities, storm drains, inlets, manholes, culverts, swales, and channels; or as otherwise approved.

These Drainage Criteria shall be used for modification of existing drainage facilities and/or construction of new facilities to improve capacity and/or alter drainage facility alignment in order to convey design flows and meet project design criteria. If existing drainage patterns are changed due to project design, then the designer shall design a solution that does not adversely impact property owners outside the project site.

Presented in these Drainage Criteria are **minimum** design and technical criteria for analysis and design of storm drainage facilities for both quantity and quality during and following land disturbing and construction activities. All new development or redevelopment projects, construction or grading projects, demolition, or any disturbance of existing ground surface shall comply with these Drainage Criteria.

All projects submitted for approval under the provisions of the City Subdivision Code shall provide adequate analysis and design of drainage systems for both water quantity and water quality during and after construction in accordance with these Drainage Criteria. Implementing facilities that go beyond the minimum is encouraged. The applicant may suggest alternatives to the provisions of these Drainage Criteria. The applicant shall be required to show that alternatives submitted for review are equal or better. These Drainage Criteria do not apply to drainage facilities in place or under construction at the time of Drainage Criteria adoption.

1.2 PRINCIPLES FOR STORM DRAINAGE PLANNING

Site development and construction must include storm drainage facilities to benefit the Owner/Applicant, other residents and users of public facilities, and limit drainage-related hazards within and outside of a project site. In designing storm drainage system improvements, the designer shall achieve the required stormwater conveyance through the project while minimizing future operation and maintenance costs, public inconvenience, flood damages, and environmental quality impacts during construction and for permanent installations. Specifically, the designer shall design temporary and permanent drainage facilities, modifications, and improvements for a project site to:

A. Provide convenience and safety to roadway and open space users in accordance with applicable drainage design criteria.

B. Protect adjacent property owners against drainage-related damages resulting from a project by maintaining or improving existing drainage patterns and conditions.

C. Facilitate site development by planning and coordinating the phasing of construction of drainage improvements to continuously protect project elements during construction.

D. Mitigate potential impacts to wetlands, riparian areas, and surface water quality.

Stormwater management involves more than conveying rainfall runoff from the land. The storm drainage system is for transmission of natural surface water and uncontaminated groundwater only. Storm drainage designs must consider impacts to vegetation, public safety, facility maintenance access and costs, and disturbances during construction. No pollutants, trash, or other deleterious substances shall be placed in the storm drain system or area tributary to the system. No industrial waste or process waste of any type shall be discharged into the storm drainage system.

1.3 DRAINAGE BASIN MASTER PLANS

The City has developed detailed drainage basin master plans that set forth requirements for new developments and identify required public improvements. The City shall give careful consideration to implementing the recommendations of drainage basin master plans, within the context of available public funds and overall priorities specified in the *City of Gillette Comprehensive Plan* (and as amended).

Prior to implementing drainage basin master plan recommendations based on modeling, the City will perform reasonableness checks of submitted modeling results based on site observations and other information (e.g., maintenance records, flooding problems due to existing pipe size, internal modeling results), where such information is reasonably available. In areas with known drainage or water quality problems, additional analysis and/or definition of additional facilities to prevent compounding of these problems must be completed.

All new development and redevelopment projects must design and construct drainage improvements that drain to an acceptable outfall in accordance with the City of Gillette-approved Final Drainage Report for the minor system (as described in the City of Gillette Design Standards, Section 101.21, Final Submittal Requirements) and the applicable master drainage plan for the major drainage system. Where no approved master drainage basin master plan exists, the applicant must prepare and obtain approval for a master drainage plan for the affected area.

1.4 RELATIONSHIP TO OTHER STANDARDS

All storm drainage improvements within the City shall be designed and constructed according to these Drainage Criteria, the City of Gillette Design Standards, the City of Gillette Standard

Construction Specifications and other City regulations and ordinances as they apply to storm drainage. These Drainage Criteria provide guidance as to the minimum requirements for storm drainage improvements. Depending on specific site conditions, storm drainage systems may need to exceed these minimum standards in order to provide adequate protection from flooding. Design methods other than those described in these Drainage Criteria shall be applied only after requesting and receiving approval according to the variance procedures defined herein. Should these design standards conflict with the Design Guide, the requirements of these Drainage Criteria shall govern. Should these Drainage Criteria conflict with the requirements of other agencies, the designer shall seek direction from the City Engineer to determine the appropriate standard to apply.

Multiple local, state and federal laws and regulations may apply when addressing stormwater management. The designer of storm drain systems must consider all laws and regulations that affect the design of each project. These include, but are not limited to the City Stormwater Ordinance, Subdivision Regulations, Floodplain Ordinance, Wyoming Department of Transportation (WYDOT) standards, Wyoming Department of Environmental Quality (WDEQ) regulations or United States Army Corps of Engineers (USACE) Permits, and the Gillette Design Standards.

If the state or federal government imposes stricter criteria, standards, or requirements, these shall apply in addition to these Drainage Criteria.

1.5 REGULATORY FLOODPLAINS

Regulatory floodplains within the City have been defined and adopted. Proposed improvements that will modify established floodplains must comply with the City's Floodplain Ordinance and all requirements of the National Flood Insurance Program.

1.6 REVIEW AND ACCEPTANCE

1.6.1 Submittal Requirements

Drainage reports shall be submitted and approved as required by the City regulations. All submitted reports shall follow the outlines provided within this section of the Drainage Criteria and be clearly and cleanly reproduced. Photo static copies of charts, tables, nomographs, calculations, or any other referenced material shall be legible. Washed out, blurred, or unreadable portions of the report are unacceptable and could warrant resubmittal of the report. The submittal shall be reviewed for adequacy based on City guidelines for report content. Incomplete or absent information may result in the report being rejected for review.

A pre-application consultation is suggested for all applicants. The applicant shall consult with City Staff for general information regarding subdivision regulations, required procedures, possible drainage problems and specific submittal requirements.

1.6.1.1 Drainage Reports

All land development or re-development projects affecting drainage or the potential for erosion and sedimentation shall submit documentation addressing the proposed changes and providing measures intended to mitigate the impacts of the proposed project to drainage facilities and downstream properties. Submittals shall be provided with the appropriate type of land use application including annexation, planning, zoning, platting and site development.

A. Preliminary Drainage Report – The purpose of the Drainage Report is to identify and/or refine conceptual solutions to the problems which may occur on-site and off-site as a result of the development or any phase of the development. The Drainage Report shall be submitted during the subdivision process with the application for Preliminary Plat. If the Zoning and Preliminary Plat applications are submitted concurrently, the submittal requirements of this section shall apply. In addition, those problems that exist on-site prior to development must be addressed during the preliminary phase.

Preliminary drainage reports shall provide an appropriate level of detail to address drainage issues and present the overall plan for the property. The report shall be based on the following outline and include appropriate background information, supporting data, calculations and plan drawing(s).

TITLE PAGE

1. Type of Report
2. Project Name
3. Prepared for
4. Prepared by
5. Date
6. P.E. Seal and Signature

INTRODUCTION

1. Location
   a. City, County, State Highway and local streets within and adjacent to the site, or the area to be served by the drainage improvements.
   b. Township, range, section, ¼ section
   c. Names of surrounding developments, properties or landmarks.
2. Description of Property
   a. Area in acres
   b. Ground cover (type of ground cover and vegetation)
   c. Existing land uses
   d. Topographic features, steepness of slopes
   e. Major drainage ways and receiving channels
   f. Existing drainage facilities
   g. Flood Hazard Zones
   h. Geologic Features (if applicable)
   i. Previous drainage studies for the property (if any)

3. Proposed Project Description
    a. Land uses
    b. Changes to existing facilities
    c. Changes to floodplains
    d. Proposed system improvements
    e. Right-of-way conveyance or acquisition required
4. Drainage Criteria
    a. Application Standards or exceptions
    b. Minor and Major Storm Frequencies
    c. Hydrologic Methods
        1. Rainfall
        2. Design Storms
        3. Runoff methods and computer models
    d. Hydraulic Methods
        1. Design standards
        2. Hydraulic models
        3. Detention Pond sizing
    e. State or Federal Regulations (if applicable).

HISTORIC DRAINAGE SYSTEM

Major Basin Description

a. Reference to major drainage way planning studies such as flood hazard delineation report, major drainage way planning reports, and flood insurance rate maps.
b. Major basin drainage characteristics and structures, existing and planned land uses within the basin.
c. Summary of off-site and on-site basin characteristics and runoff rates.

PROPOSED DRAINAGE SYSTEM

Design Concepts

a. Discussion of concept and typical drainage patterns.
b. Discussion of compliance with off-site runoff considerations.
c. Discussion of proposed drainage patterns and improvements including streets, storm sewer, culverts, open channels and detention storage.
d. Discussion of the content of tables, charts, figures, plates, or drawings presented in the report.

SUMMARY

1. Relation to off-site conditions.
2. Summary of proposed improvements.
   a. Storm sewer
   b. Culverts
   c. Open channels
   d. Detention Storage
3. Floodplain impacts.
4. State or Federal regulations.
5. Compliance with applicable regulations and standards.

REFERENCES

Reference all criteria, master plans, and technical information used in support of concepts and calculations.

APPENDICES

Background Data

a. Floodplain maps.
b. Applicable reports or report excerpts.
c. Key correspondence with adjacent property owners or utilities.

PRELIMINARY REPORT DRAWING CONTENTS

General Location Map: All drawings submitted as back-up materials with the Preliminary Plat shall be 11” x 17” in size. A map shall be provided in sufficient detail to identify drainage flows entering and leaving the development and general drainage patterns. The map shall identify any major facilities from the property (i.e., development, existing detention facilities, culverts, storm sewers) along the flow path to the nearest major drainage way.

Floodplain Information: The location of the subject property shall be included with the report. All major drainage ways shall have the floodplain defined and shown on the report drawings.

Drainage Plan: Map(s) of the proposed development at a scale of 1”=20’ to 1” = 60’ on an 11” x 17” drawing shall be included. The plan shall show the following:

1. Existing topographic contours at two (2) feet maximum intervals. The contours shall extend a minimum of one-hundred (100) feet beyond the property lines.
2. All existing drainage facilities.
3. Approximate flooding limits based on available information

4. Conceptual major drainage facilities including detention basins, storm sewers, swales, riprap, and outlet structures in the detail consistent with the proposed development plan.
5. Major drainage boundaries and sub-basin boundaries.
6. Any off-site features influencing development.
7. Proposed flow directions and, if available, proposed contours.
8. Legend to define map symbols.
9. Title block in lower right corner.

B. Final Drainage Report – The purpose of the Final Drainage Report is to present the design details for the drainage facilities discussed in the Preliminary Drainage Report. Also, any change to the Preliminary concept must be presented and fully explained.

The Final Drainage Report shall be submitted with the Final Plat, Site Plan or Building Permit applications for applicable developments.

Final drainage reports shall provide an appropriate level of detail to address the drainage issues and present sizing and locations for all proposed improvements. The report shall be based on the following outline and include appropriate background information and supporting data and calculations and plan drawing(s).

TITLE PAGE

1. Type of Report
2. Project Name
3. Prepared for
4. Prepared by
5. Date
6. P.E. Seal and Signature

INTRODUCTION

1. Location
    a. City, County, State Highway and local streets within and adjacent to the site, or the area to be served by the drainage improvements.
    b. Township, range, section, ¼ section
    c. Names of surrounding developments, properties or landmarks.
2. Description of Property
    a. Area in acres
    b. Ground cover (type of ground cover and vegetation)
    c. Existing land uses
    d. Topographic features, steepness of slopes

    e. Major drainage ways and receiving channels
    f. Existing drainage facilities
    g. Flood Hazard Zones
    h. Geologic Features (if applicable)
    i. Previous drainage studies for the property (if any)
3. Proposed Project Description
    a. Land uses
    b. Changes to existing facilities
    c. Changes to floodplains
    d. Proposed system improvements
    e. Right-of-way conveyance or acquisition required
4. Drainage Criteria
    a. Application Standards or Exceptions
    b. Minor and Major Storm Frequencies
    c. Hydrologic Methods
        i. Rainfall
        ii. Design Storms
        iii. Runoff methods and computer models
    d. Hydraulic Methods
        i. Design standards
        ii. Hydraulic models
        iii. Detention Pond sizing
    e. State or Federal Regulations (if applicable).

HISTORIC DRAINAGE SYSTEM

1. Major Basin Description
    a. Reference to major drainage way planning studies such as flood hazard delineation report, major drainage way planning reports, and flood insurance rate maps.
    b. Major basin drainage characteristics and structures, existing and planned land uses within the basin.
    c. Summary of off-site and on-site basin characteristics and runoff rates.
2. Sub-Basin Description
    a. Discussions of historic drainage patterns of the property.

b. Discussion of off-site drainage flows and flow patterns and impact on development under existing and fully developed basin conditions.

c. Summary of off-site and on-site basin characteristics and runoff rates.

PROPOSED DRAINAGE SYSTEM

1. Design Concepts

   a. Discussion of concept and typical drainage patterns.

   b. Discussion of compliance with off-site runoff considerations.

   c. Discussion of proposed drainage patterns and improvements including streets, storm sewer, culverts, open channels and detention storage.

   d. Discussion of the content of tables, charts, figures, plates, or drawings presented in the report.

2. Design Details

   a. Discussion of drainage problems encountered and solutions at specific design points.

   b. Discussion of detention storage and outlet design.

   c. Discussion of maintenance and access aspects of the design.

   d. Discussion of impacts of concentrating the flow on the downstream properties.

   e. Summary of basin characteristics and runoff rates.

SUMMARY

1. Relation to off-site conditions.
2. Summary of proposed improvements.

   a. Storm sewer

   b. Culverts

   c. Open channels

   d. Detention Storage

3. Floodplain impacts.
4. State or Federal regulations.
5. Compliance with applicable regulations and standards.

REFERENCES

Reference all criteria, master plans, and technical information used in support of concepts and calculations.

APPENDICES

1. Background Data

a. Floodplain maps.

b. Applicable reports or report excerpts.

c. Key correspondence with adjacent property owners or utilities.

2. Hydrologic Computations

   a. Land uses regarding adjacent properties.

   b. Soil types, coverage and loss coefficients

   c. Proposed land uses for project by basin.

   d. Time of concentration and runoff coefficients for each basin.

   e. Basin parameters used for modeling including basin area, length, slope, distance to centroid and routing elements.

   f. Initial and major storm runoff at specific design points for off-site and on-site flows.

   g. Off-site, historic and fully developed runoff computations at specific design points

   h. Hydrographs at critical design points.

   i. Schematic diagram of hydrology model showing basins and routing elements and combination elements.

3. Hydraulic Computations

   a. Culvert Capacities and inlet and outlet protection.

   b. Storm sewer capacity, including energy grade line (EGL) and hydraulic grade line (HGL) elevations.

   c. Gutter capacity as compared to allowable.

   d. Storm inlet capacity including roughness coefficients, trickle channels, freeboard, hydraulic grade line, and slope protection.

   e. Check and/or channel drop placement.

   f. Detention area volume capacity and outlet capacity calculations; depths of detention basins, outlet configuration.

   g. Downstream/outfall capacity to the Major Drainage way system.

4. Miscellaneous Information

   a. Other documents relating to drainage conditions on the property.

   b. Agreements with property owners or other agencies.

   c. Permits, etc.

FINAL REPORT DRAWING CONTENTS

General Location Map: All drawings submitted as back-up materials for a Permit To Construct (PTC) shall be 11” x 17” in size. A map shall be provided in sufficient detail to identify drainage flows entering and leaving the development and general drainage patterns. The map shall identify any major construction (i.e., development, existing detention facilities, culverts, storm sewers) along the entire path of drainage. Basins and divides are to be identified and topographic contours are to be included.

Floodplain Information: The location of the subject property shall be included with the report. All major drainage ways shall have the floodplain defined and shown on the report drawings.

Drainage Plan: Map(s) of the proposed development at a scale of 1”=20’ to 1”=60’ on an 11” x 17” drawing shall be included. The plan shall show the following:

1. Existing topographic contours at two (2) foot maximum intervals. The contours shall extend a minimum of one-hundred (100) feet beyond the property lines and as necessary to show relevant off-site drainage features.
2. Location, elevation, and description of City of Gillette benchmarks.
3. Existing and Proposed property lines and easements with purpose noted. Streets, indicating names, ROW width, flowline width, curb type, sidewalk, and approximate slopes.
4. Existing drainage facilities and structures, including roadside ditches, crosspans, drainage ways, gutter flow direction, and culverts. All pertinent information such as material, size, shape, slope, and location shall also be included.
5. Overall drainage area boundary and drainage sub-area boundaries.
6. Proposed type of street flow (i.e., vertical or combination curb and gutter), roadside ditch, gutter, slope and flow directions, and crosspans.
7. Proposed storm sewers and open drainage ways, including inlets, manholes, culverts, and other appurtenances, including riprap protection and profiles showing existing and proposed sizes and grades and hydraulic grade lines for minor and major storms.
8. Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties.
9. Proposed storm water quality facilities.
10. Routing and accumulation of flows at various critical points for the minor storm event listed on the drawing.
11. Routing and accumulation of flows at various critical points for the major storm event listed on the drawing.
12. Volume and release rates for detention storage facilities and outlet works details with a site plan showing proposed grading, emergency spillway and details
13. Location and elevation of all existing floodplain affecting the property.
14. Location and elevation of all existing and proposed utilities affected by or affecting the drainage design.
15. Routing of off-site drainage flow through the development.

16. Definition of flow path leaving the development through the downstream properties ending at a major drainage way.
17. Typical sections for open channels and cross sections showing existing proposed grades, side slopes.
18. Legend to define map symbols.
19. Title block in lower right corner.
20. Seal of Professional Engineer licensed to practice in Wyoming.
21. Standard Notes.

1.6.1.2 Stormwater Permit and Stormwater Pollution Prevention Plan

A Stormwater Pollution Prevention Plan (SWPPP) must also be developed per Section 11.4 of these Drainage Criteria and submitted to obtain a stormwater permit. Site planning and drainage planning should, whenever possible, occur concurrently with site grading and erosion control planning.

Implementation and maintenance of erosion control measures are ultimately the responsibility of the property owner. Should the approved SWPPP not function as intended, and it is determined by the City that additional measures are needed, the owner will have to provide additional measures needed to reduce soil erosion and sediment discharged from the construction site.

1.6.2 Variances

Variances from these criteria may be warranted by site conditions or engineering judgment. All variances shall be explained in writing and approved by the City.

The City Engineer may, at his or her sole discretion, waive or grant variances from requirements contained in these Drainage Criteria upon a finding of undue hardship and no significant deleterious effects to public safety, health, welfare, and the environment, provided that the project complies with the requirements of Gillette’s Stormwater Ordinance. Variances from these Drainage Criteria will be considered on a case-by-case basis. Variance requests must be submitted to the Engineering Division in writing, and at a minimum, must contain the following information:

1. Identify criteria for which a waiver or variance is requested.
2. Explain why the criteria cannot be met.
3. Define alternative standard(s) to meet the intent of the criteria.
4. Provide supporting documentation, necessary calculations, and other relevant information supporting the request.
5. Sign and seal the variance request (by an engineer licensed in the State of Wyoming).

SECTION TWO
RAINFALL

2.1 INTRODUCTION

Design rainfall data are required for use in completing hydrologic analyses as described in Section 3, Runoff Analysis, of these Drainage Criteria. Presented in this section are: 1) design storm frequencies for the City, 2) information on the NRCS Rainfall Distributions, and 3) an intensity-duration-frequency table for use with the Rational Method. All hydrological analyses within City shall use the rainfall data presented herein for calculating storm runoff. There may be cases where the designer needs to consider events more extreme than the 100-year storm (e.g., for public safety).

The design storms and intensity-frequency-duration tables for the City were developed using the rainfall data and procedures presented in the NOAA Atlas 2, "Precipitation-Frequency Atlas of the Western United States," Volume II, Wyoming, 1973 and are presented herein for convenience.

2.2 DESIGN STORM FREQUENCIES

Storm drainage improvements shall be designed to convey "Minor" and "Major" storm frequencies as defined in Table 2-1.

Table 2-1
Design Storm Frequency by Zoning District/Land Use

Zoning District/Land Use[1]	Design Storm Frequency (Return Period, Year)	
	Minor	Major
Residential	2	100
Commercial/Office/Business	5	100
Industrial/Central Business	10	100
Parks, Cemeteries, Open Public Land	2	100

[1] Where multiple zoning designations and land uses apply within the same drainage basin, the greater design storm shall govern.

All drainage improvements shall be designed to convey the Minor storm, as a minimum. Minor facility improvements may need to be designed to carry part or all of the Major storm flows if Major storm flows cannot be conveyed safely to a suitable receiving system or if allowable flow depths in streets cannot be maintained. In addition, consideration should be given to the impact of storms greater than the Major storm to anticipate and reduce the potential for significant flood damage.

2.3 NATURAL RESOURCES CONSERVATION SERVICE (NRCS) DESIGN STORMS

The NRCS Type II 24-hour rainfall distribution as shown in Table 2-2 shall be used for the City. More information on this distribution and unit hydrograph procedures can be found in the NRCS publication "Urban Hydrology for Small Watersheds" (TR-55).

Table 2-2
NRCS 24-Hour Type II Rainfall Distribution
(Values are in Percent of Total 24-Hour Precipitation Depth)

Hour	0.1-Hour Intervals									
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0	0.1	0.2	0.3	0.41	0.51	0.62	0.72	0.83	0.94
1	1.05	1.16	1.27	1.38	1.5	1.61	1.73	1.84	1.96	2.08
2	2.2	2.32	2.44	2.57	2.69	2.81	2.94	3.06	3.19	3.32
3	3.45	3.58	3.71	3.84	3.98	4.11	4.25	4.39	4.52	4.66
4	4.8	4.94	5.08	5.23	5.38	5.53	5.68	5.83	5.98	6.14
5	6.3	6.46	6.62	6.79	6.96	7.12	7.3	7.47	7.64	7.82
6	8	8.18	8.36	8.55	8.74	8.92	9.12	9.31	9.5	9.7
7	9.9	10.1	10.3	10.51	10.72	10.93	11.14	11.35	11.56	11.78
8	12	12.22	12.46	12.7	12.96	13.22	13.5	13.79	14.08	14.38
9	14.7	15.02	15.34	15.66	15.98	16.3	16.63	16.97	17.33	17.71
10	18.1	18.51	18.95	19.41	19.89	20.4	20.94	21.52	22.14	22.8
11	23.5	24.27	25.13	26.09	27.15	28.3	30.68	35.44	43.08	56.79
12	66.3	68.2	69.86	71.3	72.52	73.5	74.34	75.14	75.88	76.56
13	77.2	77.8	78.36	78.9	79.42	79.9	80.36	80.8	81.22	81.62
14	82	82.37	82.73	83.08	83.42	83.76	84.09	84.42	84.74	85.05
15	85.35	85.65	85.94	86.22	86.49	86.76	87.02	87.28	87.53	87.77
16	88	88.23	88.45	88.68	88.9	89.12	89.34	89.55	89.76	89.97
17	90.18	90.38	90.58	90.78	90.97	91.17	91.36	91.55	91.73	91.92
18	92.1	92.28	92.45	92.63	92.8	92.97	93.13	93.3	93.46	93.62
19	93.77	93.93	94.08	94.23	94.38	94.52	94.66	94.8	94.93	95.07
20	95.2	95.33	95.46	95.59	95.72	95.84	95.97	96.1	96.22	96.35
21	96.47	96.6	96.72	96.85	96.97	97.09	97.22	97.34	97.46	97.58
22	97.7	97.82	97.94	98.06	98.18	98.29	98.41	98.53	98.64	98.76
23	98.87	98.99	99.1	99.22	99.33	99.44	99.56	99.67	99.78	99.89
24	100									

2.4 INTENSITY – DURATION – FREQUENCY (IDF) FOR RATIONAL METHOD

Rainfall depths, intensities and storm durations summarized herein shall be applied to all projects within the City to estimate rates and volumes of rainfall runoff. Rainfall depths for the City are summarized in Table 2-3.

Table 2-3
Rainfall Depths (in), Durations and Frequencies

Storm Frequency (Return Period, year)	Storm Duration (hours)	
	1	24
2	0.83	1.60
5	1.25	2.20
10	1.50	2.60
25	1.80	3.20
50	2.05	3.60
100	2.32	4.00

Source: NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume II, Wyoming, 1973

Listed in Table 2-4 are the rainfall intensity-duration values calculated for use with the Rational Method in small watersheds that are 160 acres or less in size, based on Equation 2.1:

$$I = (28.5\ P_1) / (10+ T_c)^{0.786} \qquad (2.1)$$

Where:

I = rainfall intensity (in/hr)
P_1 = 1-hour point rainfall depth (in)
T_c = time of concentration (min)

Source: UDFCD Drainage Criteria Manual, Volume I, 2006

Equation 2.1 is applicable for durations between and including 5 and 59 minutes.

Table 2-4
Rainfall Intensity-Duration-Frequency Data

Storm Frequency Duration/Tc (min)	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
5	2.82	4.24	5.09	6.11	6.95	7.87
6	2.68	4.03	4.84	5.80	6.61	7.48
7	2.55	3.84	4.61	5.53	6.30	7.13
8	2.44	3.67	4.41	5.29	6.02	6.82
9	2.34	3.52	4.23	5.07	5.77	6.53
10	2.25	3.38	4.06	4.87	5.55	6.28
11	2.16	3.25	3.91	4.69	5.34	6.04
12	2.08	3.14	3.77	4.52	5.15	5.82
13	2.01	3.03	3.64	4.36	4.97	5.62
14	1.95	2.93	3.52	4.22	4.81	5.44
15	1.88	2.84	3.41	4.09	4.65	5.27
16	1.83	2.75	3.30	3.96	4.51	5.11
17	1.77	2.67	3.21	3.85	4.38	4.96
18	1.72	2.60	3.12	3.74	4.26	4.82
19	1.68	2.53	3.03	3.64	4.14	4.69
20	1.63	2.46	2.95	3.54	4.03	4.56
21	1.59	2.40	2.88	3.45	3.93	4.45
22	1.55	2.34	2.80	3.37	3.83	4.34
23	1.51	2.28	2.74	3.29	3.74	4.23
24	1.48	2.23	2.67	3.21	3.65	4.14
25	1.45	2.18	2.61	3.14	3.57	4.04
26	1.41	2.13	2.56	3.07	3.49	3.95
27	1.38	2.09	2.50	3.00	3.42	3.87
28	1.36	2.04	2.45	2.94	3.35	3.79
29	1.33	2.00	2.40	2.88	3.28	3.71
30	1.30	1.96	2.35	2.82	3.22	3.64
31	1.28	1.92	2.31	2.77	3.15	3.57
32	1.25	1.89	2.26	2.72	3.10	3.50
33	1.23	1.85	2.22	2.67	3.04	3.44
34	1.21	1.82	2.18	2.62	2.98	3.38
35	1.19	1.79	2.15	2.57	2.93	3.32
36	1.17	1.76	2.11	2.53	2.88	3.26
37	1.15	1.73	2.07	2.49	2.83	3.21
38	1.13	1.70	2.04	2.45	2.79	3.15
39	1.11	1.67	2.01	2.41	2.74	3.10

Table 2-4
Rainfall Intensity-Duration-Frequency Data

Storm Frequency Duration/Tc (min)	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
40	1.09	1.65	1.97	2.37	2.70	3.05
41	1.08	1.62	1.94	2.33	2.66	3.01
42	1.06	1.60	1.91	2.30	2.62	2.96
43	1.04	1.57	1.89	2.26	2.58	2.92
44	1.03	1.55	1.86	2.23	2.54	2.88
45	1.01	1.53	1.83	2.20	2.50	2.83
46	1.00	1.51	1.81	2.17	2.47	2.79
47	0.99	1.48	1.78	2.14	2.43	2.76
48	0.97	1.46	1.76	2.11	2.40	2.72
49	0.96	1.44	1.73	2.08	2.37	2.68
50	0.95	1.43	1.71	2.05	2.34	2.65
51	0.93	1.41	1.69	2.03	2.31	2.61
52	0.92	1.39	1.67	2.00	2.28	2.58
53	0.91	1.37	1.65	1.98	2.25	2.55
54	0.90	1.36	1.63	1.95	2.22	2.52
55	0.89	1.34	1.61	1.93	2.20	2.49
56	0.88	1.32	1.59	1.91	2.17	2.46
57	0.87	1.31	1.57	1.88	2.14	2.43
58	0.86	1.29	1.55	1.86	2.12	2.40
59	0.85	1.28	1.53	1.84	2.10	2.37
60	0.83	1.25	1.50	1.80	2.05	2.32

Source: NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United State, Volume II, Wyoming, 1973 and rainfall intensity equation 2.1.

Rainfall losses shall be based on land use and soil types. Soil types according the NRCS hydrologic soil group designations (A, B, C and D) shall be used to determine infiltration rates. More information on losses and runoff is included in Section 3, Runoff Analysis.

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SECTION THREE
RUNOFF ANALYSIS

3.1 INTRODUCTION

This section presents acceptable methods for calculation of runoff and estimation of peak flow rates. Other methods may be approved on a case-by-case basis, after submittal of a written request to the City that includes an explanation of why the different method is required.

3.2 RUNOFF CALCULATION METHODS

Acceptable runoff calculation methods are listed in Table 3-1.

Table 3-1
Runoff Calculation Methods Acceptable for Use in Gillette

Runoff Calculation Method	Application	Area Limit	Use Notes and Reference
Rational Method	Rational Method is appropriate for small on-site detention designs. Should not be used when routing of hydrographs is required.	Simple catchments less than 160 acres in size.	Use Gillette's standard forms for the calculation of Time of Concentration and Storm Drainage System Design. (Reference Chapter 05, Volume I of the USDCM for use.)
NRCS-UH	Use NRCS-UH in combination with SWMM when routing of hydrographs is required.	Required for basins greater than 160 acres in size. Can be used for basins greater than 20 acres in size, and for catchments 5-20 acres in size with smaller unit hydrograph time.	See Section 3.4 below. Use the Land Use maps available from the City. Original Source TR-55 "Urban Hydrology for Small Watersheds", (SCS 1986) Updated Source NRCS National Engineering Handbook, Part 630 (NRCS date varies)
Published hydrologic information	May be used where Gillette has developed detailed hydrologic studies appropriate for use in the study area.	N/A	Use values in published reports unless a compelling reason to modify published values can be demonstrated to the City.

The Rational Method, the NRCS Unit Hydrograph Method (NRCS UH), or published hydrologic information shall be applied to all projects within Gillette to develop peak rates of runoff and runoff hydrographs for the design of storm drainage improvements. Generally, these methods shall be applied as described in the method documentation. Estimates of runoff shall be made for Minor and Major storm events as needed for the type of development and drainage facility being designed.

The Rational Method is generally used for smaller catchments when only the peak flow rate or the total volume of runoff is needed (e.g., storm drain sizing or simple detention basin sizing).

The NRCS UH is used for larger catchments and when a hydrograph of the storm event is needed (e.g., sizing large detention facilities). However, NRCS UH can be used for smaller basins to determine peak flow rates and when a hydrograph is needed.

All hydrologic methods require the user to estimate watershed characteristics (i.e., area, length, slope, imperviousness, and the characteristic response time for the watershed (time of concentration). The NRCS UH method also requires calculation of the precipitation losses. Procedures for use of these are described in the program user's manuals, and the following paragraphs.

3.2.1 Use of Modeling Software and Design Spreadsheets

Computer software programs, hydrologic models, and spreadsheets referenced in these Drainage Criteria are design aids that may be useful in designing drainage improvements. Use of these design aids is not a substitute for application of sound engineering judgment and proper engineering techniques.

Although the design aids recommended in these Drainage Criteria have been developed using a high standard of care, it is possible that some nonconformities, defects, bugs, and errors with the software programs will be discovered as they become more widely used. The City does not warrant that any version of these design aids will be error free or applicable to all conditions encountered by the designer, and the City shall not be held liable for their use.

Table 3-2 provides a brief description of the generally available computer software programs that may be used in the City, and their typical requirements for use. For more information on each of these packages, the designer should refer to the most recent vendor documentation. Other software packages may be used with prior written consent from the City Engineer.

Table 3-2
Requirements for Software use in Gillette

Software Application	Application	Requirements for Use
HEC-HMS	Used to abstract storm runoff hydrographs and route and combine hydrographs for sub-catchments. Appropriate for use in more complex basins.	Use the NRCS CN number option available under abstraction methods. Provide a copy of input/output listings for the model and an electronic copy of the modeling results in the Final Drainage Report submittal. A summary table of peak flow rates at design points should be included in the Final Drainage Report submittal.
EPA SWMM	Used to route and combine hydrographs for more complex basins.	Use hydrographs developed with the NRCS UH method. Provide a copy of input/output listings for the model and an electronic copy of the modeling results in the Final Drainage Report submittal. A summary table of peak flow rates at design points should be included in the Final Drainage Report submittal.

Table 3-2
Requirements for Software use in Gillette

Software Application	Application	Requirements for Use
Storm-CAD	Used to design storm drain systems with multiple inlets.	Use steady peak flow rates from the approved methods. Provide a copy of input/output listings for the model and an electronic copy of the modeling results in the Final Drainage Report submittal. A summary table of peak flow rates at design points should be included in the Final Drainage Report submittal.
Win TR-20	Used to abstract storm runoff hydrographs and route and combine hydrographs for sub-catchments. Appropriate for use in more complex basins.	Provide a copy of input/output listings for the model and an electronic copy of the modeling results in the Final Drainage Report submittal. A summary table of peak flow rates at design points should be included in the Final Drainage Report submittal.

3.3 RATIONAL METHOD

For watersheds that are not complex and have total areas less than 160 acres, the design storm runoff may be analyzed using the Rational Method. This method is widely used due to its simplicity and level of general acceptance. The Rational Method is based on the formula:

$$Q = CIA \tag{3.1}$$

Where:

Q = Maximum rate of runoff in cubic feet per second (cfs)
C = Runoff coefficient
I = Average intensity of rainfall (in/hr) based upon T_c (Eq. 2.1)
A = Contributing watershed area (acre).

The basic assumptions made when applying the Rational Method are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.
2. The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

The maximum runoff rate occurs when the entire area is contributing flow.

The rainfall intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration, t_c, which shall be calculated as described in Section 3.3.1. Rainfall intensities are provided in Section 2.4.

3.3.1 Time of Concentration

The time of concentration, t_c, is the time required for runoff to flow from the most remote part of the watershed area to the point of interest. The time of concentration is calculated so that the average rainfall rate for a corresponding duration can be determined from the rainfall intensity-duration-frequency curves. Time of concentration consists of an initial time or overland flow time, t_i, plus travel (or channel flow) time, t_t, which is in a combined form. In both urban and non-urban environments, the initial or overland flow is assumed to occur as sheet flow and is a function of surface cover and slope, depression storage, antecedent rainfall, and infiltration capacity of the soil, with a maximum limit of 300 ft.

The time of concentration for both urban and non-urban areas is calculated as follows:

$$t_c = t_i + t_t \qquad (3.2)$$

Where:

t_c = Time of concentration (min)
t_i = Initial, Inlet, or overland flow time (min)
t_t = Travel time in the ditch, channel, gutter, storm drain, etc. (min)

The initial or overland flow time, t_i, may be calculated using Equation 3.3:

$$t_i = 1.8\,(1.1 - C_5)\,L_o^{1/2} / S^{1/3} \qquad (3.3)$$

Where:

t_i = Initial or Overland Flow Time (min)
C_5 = 5-year Runoff Coefficient
L_o = Length of Overland Flow, (ft, 300-ft maximum)
S = Average Watershed Slope (percent)

The equation for initial time was originally developed by the Federal Aviation Administration (FAA 1970) for use with the Rational Method. However, this equation is also valid for computation of the initial or overland flow time for the NRCS UH Method for small basins. The 5-year runoff coefficient, C_5, is listed in Table 3-3 as a function of NRCS Hydrologic Soil Group.

Table 3-3
Runoff Coefficients, C

Percentage Imperviousness	NRCS Hydrologic Soil Groups Type C and D					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.04	0.15	0.25	0.37	0.44	0.5
5%	0.08	0.18	0.28	0.39	0.46	0.52
10%	0.11	0.21	0.3	0.41	0.47	0.53
15%	0.14	0.24	0.32	0.43	0.49	0.54
20%	0.17	0.26	0.34	0.44	0.5	0.55
25%	0.2	0.28	0.36	0.46	0.51	0.56
30%	0.22	0.3	0.38	0.47	0.52	0.57
35%	0.25	0.33	0.4	0.48	0.53	0.57
40%	0.28	0.35	0.42	0.5	0.54	0.58
45%	0.31	0.37	0.44	0.51	0.55	0.59

Table 3-3
Runoff Coefficients, C

Percentage Imperviousness	NRCS Hydrologic Soil Groups Type C and D					
50%	0.34	0.4	0.46	0.53	0.57	0.6
55%	0.37	0.43	0.48	0.55	0.58	0.62
60%	0.41	0.46	0.51	0.57	0.6	0.63
65%	0.45	0.49	0.54	0.59	0.62	0.65
70%	0.49	0.53	0.57	0.62	0.65	0.68
75%	0.54	0.58	0.62	0.66	0.68	0.71
80%	0.6	0.63	0.66	0.7	0.72	0.74
85%	0.66	0.68	0.71	0.75	0.77	0.79
90%	0.73	0.75	0.77	0.8	0.82	0.83
95%	0.8	0.82	0.84	0.87	0.88	0.89
100%	0.89	0.9	0.92	0.94	0.95	0.96
Percentage Imperviousness	NRCS Hydrologic Soil Groups Type B					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.02	0.08	0.15	0.25	0.3	0.35
5%	0.04	0.1	0.19	0.28	0.33	0.38
10%	0.06	0.14	0.22	0.31	0.36	0.4
15%	0.08	0.17	0.25	0.33	0.38	0.42
20%	0.12	0.2	0.27	0.35	0.4	0.44
25%	0.15	0.22	0.3	0.37	0.41	0.46
30%	0.18	0.25	0.32	0.39	0.43	0.47
35%	0.2	0.27	0.34	0.41	0.44	0.48
40%	0.23	0.3	0.36	0.42	0.46	0.5
45%	0.26	0.32	0.38	0.44	0.48	0.51
50%	0.29	0.35	0.4	0.46	0.49	0.52
55%	0.33	0.38	0.43	0.48	0.51	0.54
60%	0.37	0.41	0.46	0.51	0.54	0.56
65%	0.41	0.45	0.49	0.54	0.57	0.59
70%	0.45	0.49	0.53	0.58	0.6	0.62
75%	0.51	0.54	0.58	0.62	0.64	0.66
80%	0.57	0.59	0.63	0.66	0.68	0.7
85%	0.63	0.66	0.69	0.72	0.73	0.75
90%	0.71	0.73	0.75	0.78	0.8	0.81
95%	0.79	0.81	0.83	0.85	0.87	0.88
100%	0.89	0.9	0.92	0.94	0.95	0.96

Table 3-3
Runoff Coefficients, C

Percentage Imperviousness	NRCS Hydrologic Soil Groups Type A					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0	0	0.05	0.12	0.16	0.2
5%	0	0.02	0.1	0.16	0.2	0.24
10%	0	0.06	0.14	0.2	0.24	0.28
15%	0.02	0.1	0.17	0.23	0.27	0.3
20%	0.06	0.13	0.2	0.26	0.3	0.33
25%	0.09	0.16	0.23	0.29	0.32	0.35
30%	0.13	0.19	0.25	0.31	0.34	0.37
35%	0.16	0.22	0.28	0.33	0.36	0.39
40%	0.19	0.25	0.3	0.35	0.38	0.41
45%	0.22	0.27	0.33	0.37	0.4	0.43
50%	0.25	0.3	0.35	0.4	0.42	0.45
55%	0.29	0.33	0.38	0.42	0.45	0.47
60%	0.33	0.37	0.41	0.45	0.47	0.5
65%	0.37	0.41	0.45	0.49	0.51	0.53
70%	0.42	0.45	0.49	0.53	0.54	0.56
75%	0.47	0.5	0.54	0.57	0.59	0.61
80%	0.54	0.56	0.6	0.63	0.64	0.66
85%	0.61	0.63	0.66	0.69	0.7	0.72
90%	0.69	0.71	0.73	0.76	0.77	0.79
95%	0.78	0.8	0.82	0.84	0.85	0.86
100%	0.89	0.9	0.92	0.94	0.95	0.96

Source: USDCM 2006

The overland flow length, L_o, is generally defined as the length over which the flow characteristics appear as sheet flow or very shallow flow in broad, grassed swales. Changes in land slope, surface characteristics, and small drainage ditches or gullies will tend to force the overland flow into a combined flow condition, which results in higher flow velocities and shorter travel times. Therefore, the initial flow time is limited to the time to travel a distance of 300 feet for urban watersheds and 500 feet for non-urban watersheds.

For longer watersheds, the travel time, t_t, must be added to the overland flow time. For non-urban areas, the travel time occurs in a combined form, such as a small swale, channel, or wash. In urban areas, the travel time occurs in a combined form, such as in the storm drain, paved gutter, roadside drainage ditch, or drainage channel. Travel time can be calculated using Manning's equation and the hydraulic properties of the storm drain, gutter, ditch, or channel, or it can be approximated from Equation 3.4:

$$V = C_v \, S_w^{1/2} \qquad (3.4)$$

Where:

V = Velocity, feet per second (fps)
S_w = watercourse slope, ft/ft
C_v = Conveyance coefficient

Overland flow in urbanized watersheds can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas and can be calculated using the procedure described above. Travel time, t_t, to the first design point or inlet is often determined based on the conveyance coefficient for "Paved areas and shallow swales" (Table 3-4), but can be estimated using Manning's equation.

Table 3-4
Conveyance Coefficient, C_v

Type of Land Surface	Conveyance Coefficient, C_v
Heavy meadow	2.5
Tillage/field	5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

Source: USDCM 2006

The time of concentration for the first design point in an urbanized watershed using this procedure should not exceed the time of concentration calculated using Equation 3.5, which was developed using rainfall/runoff data collected in urbanized regions (USDCM 2006).

$$t_c = (L / 180) + 10 \tag{3.5}$$

Where:

t_c = Time of Concentration at the first design point (min)
L = Watershed Length (ft)

This method may result in a shorter time of concentration at the first design point which would govern in an urbanized watershed. The minimum t_c to the first urban design point is 5 minutes. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream reaches. The minimum recommended t_c for non-urban watersheds is 10 minutes.

3.3.2 Runoff Coefficients

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all which effect the time distribution and peak rate of runoff. Determination of the coefficient requires judgment and understanding on the part of the engineer.

Based on data collected by the UDFCD since 1969, an empirical set of relationships between C and the imperviousness percentage was developed, expressed as:

$$C_A = K_A + (1.31\, i^3 - 1.44\, i^2 + 1.135\, i - 0.012) \text{ for } C_A \geq 0, \text{ otherwise } C_A = 0$$

(3.6)

$$C_{CD} = K_{CD} + (0.858\, i^3 - 0.786\, i^2 + 0.774\, i + 0.04)$$

$$C_B = (C_A + C_{CD})/2$$

Where:

i = % imperviousness expressed as a decimal

C_A = Runoff coefficient for NRCS Type A soils

C_B = Runoff coefficient for NRCS Type B soils

C_{CD} = Runoff coefficient for NRCS Type C and D soils

K_A, K_{CD} = Correction factors for Type A, C, and D soils (Table 3-5)

Percent imperviousness, i, for various land use types are listed in Table 3-5, and may also be estimated for single family residential areas from Figure 3-1, Figure 3-2 and Figure 3-3. More information on soil types is included in Section 3.4.3, Precipitation Losses.

3.3.3 Application

The first step in applying the Rational Method is to obtain a topographic map or develop a grading plan and define the boundaries of the project site and all the relevant drainage basins. Drainage basins shall include all areas tributary to the site, including off-site areas. A field check is recommended to verify basin boundaries and flow paths, and field surveys may be required in some cases. At this stage of planning, the possibility for diversion of watershed runoff should be identified. The designer should note that the major storm watershed does not always coincide with the minor storm watershed. This is often the case in urban areas where a low-flow will stay within the gutter and follow the lowest grade, but large flows will be sufficiently deep or flowing with a high velocity such that part of the runoff will overflow street crown and flow into a new sub-watershed.

When analyzing the major runoff occurring on an area that has a storm drain system sized for the minor storm, the Rational Method must be applied with consideration to separate flow paths. Normal application of the Rational Method assumes that all of the runoff is collected by the storm drain. For the minor storm design, the time of concentration is dependent upon the flow time in the drain. However, during the major storm runoff, the drains will probably be at capacity and could not carry the additional water flowing to the inlets. This additional water then flows overland past the inlets, generally at a lower velocity than the flow in the storm drains. If a separate time of concentration analysis is made for the pipe flow and surface flow, a time lag between the surface flow peak and the pipe flow peak will occur. This lag, in effect, will allow the pipe to carry a larger portion of the major storm runoff than would be predicted using the minor storm time of concentration. The basis for this increased benefit is that the excess water from one inlet will flow to the next inlet downhill, using the overland route. If that inlet is also at capacity, the water will often continue on the surface until capacity is available in the storm drain. The analysis of this aspect of the interaction between the storm drain system and the major storm runoff is complex.

Because of the limitations of the Rational Method, the following guidelines on its application are recommended:

- The individual sub-basin sizes should not be greater than 20 acres.
- The aggregate of all sub-basin areas should not be greater than 160 acres.
- The sub-basin should be reasonably homogeneous for existing and for projected land use.
- Where basins are not homogenous, then the rational method may report a flow rate that is lower than the basins might report if subdivided. For example, a flow rate calculated for a basin that is developed in the lower end but not in the upper end (and where the C factor is averaged) may give a lower result than if just the developed area was considered.

Peak Storm Runoff Rates for Off-Site Areas

Runoff rates from off-site areas that are undeveloped but have a comprehensive development plan (comp plan) in place will be calculated using imperviousness values in Table 3-5 for the land use or surface characteristic specified by the comp plan. Runoff rates from off-site areas that are undeveloped and where proposed uses are unknown will be calculated using an imperviousness of 45 percent as specified in Table 3-5.

Runoff rates from off-site areas that are developed should be determined by calculating the actual imperviousness of the area using approved drainage reports, aerial photography, and field investigations. If this information is not available, imperviousness shall be determined based on general land use or surface characteristic as shown in Table 3-5.

The reduction of peak runoff rates from detention shall be assumed *only* for developments that are confirmed to have detention facilities and shall be subject to approval by the City. The discharge rates from these detention facilities shall be assumed to be those rates approved in the final drainage report for each off-site upstream area having detention facilities.

Table 3-5
Recommended Percentage Imperviousness Values

Land Use or Surface Characteristics	Impervious Percentage
Business:	
Commercial areas	95
Neighborhood areas	85
Residential:	
Single-family	*
Multi-unit (detached)	60
Multi-unit (attached)	75
Half-acre lot or larger	*
Apartments	80

Table 3-5
Recommended Percentage Imperviousness Values

Land Use or Surface Characteristics	Impervious Percentage
Industrial:	
Light areas	80
Heavy areas	90
Parks, cemeteries	5
Playgrounds	10
Schools	50
Railroad yard areas	15
Undeveloped areas:	
Historic flow analysis	2
Greenbelts, agricultural	2
Off-site flow analysis (when land use not defined)	45
Streets:	
Paved	100
Gravel (packed)	40
Drive and walks	90
Roofs	90
Lawns, sandy soil	0
Lawns, clayey soil	0
Water	100

*See Figures 3-1 through 3-3 for impervious percentage

Table 3-6
Correction Factors K_A and K_{CD}

NRCS Soil Type	Storm Return Period (years)					
	2	5	10	25	50	100
A	0	-0.08 i + 0.09	-0.14 i + 0.17	-0.19 i + 0.24	-0.22 i + 0.28	-0.25 i + 0.32
C and D	0	-0.10 i + 0.11	-0.18 i + 0.21	-0.28 i + 0.33	-0.33 i + 0.40	-0.39 i + 0.46

Figure 3-1
Impervious Percentage for Single-Family Residential Homes

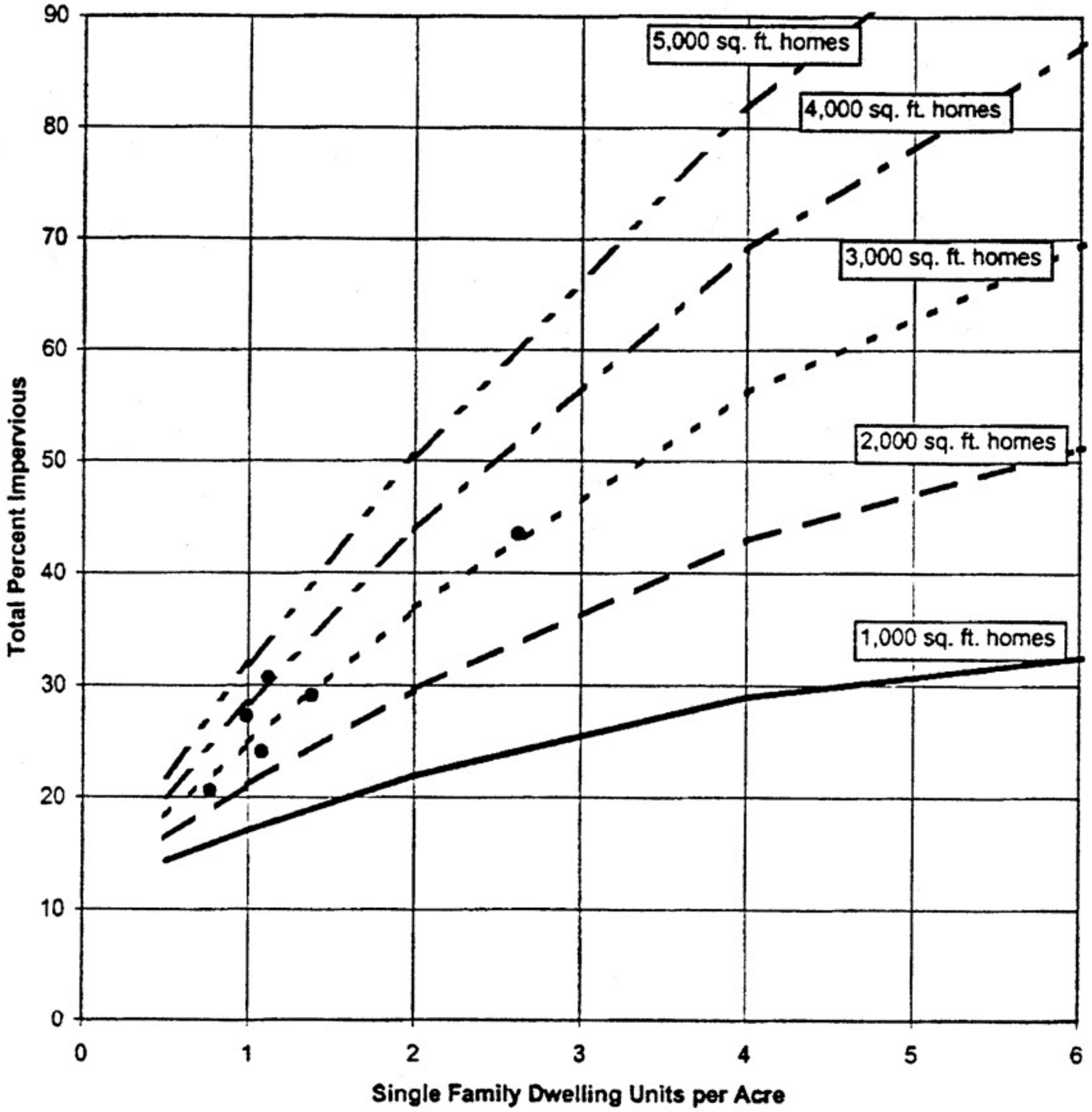


Figure 3-2
Impervious Percentage for Single-Family Residential Split-Level Homes

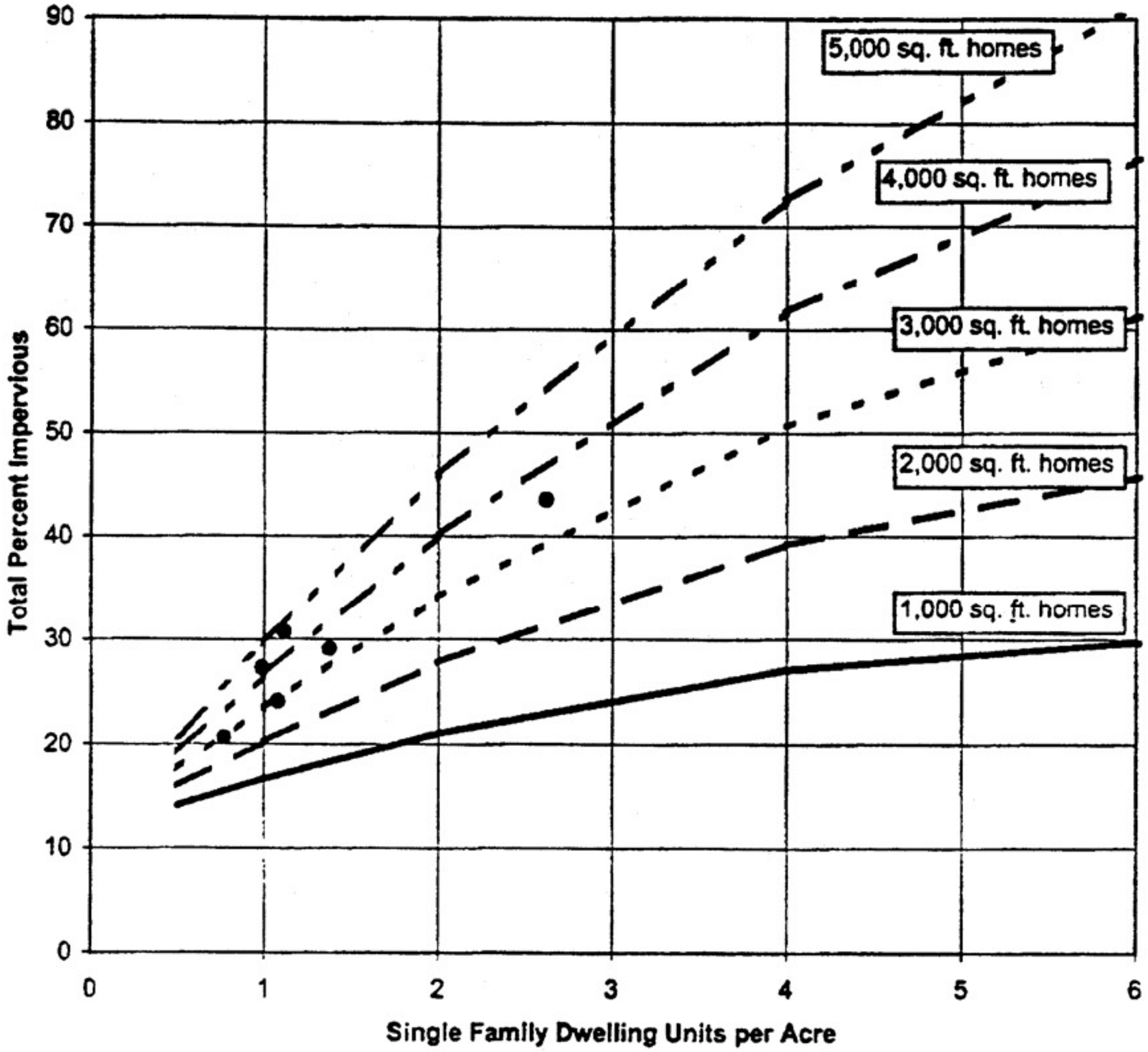
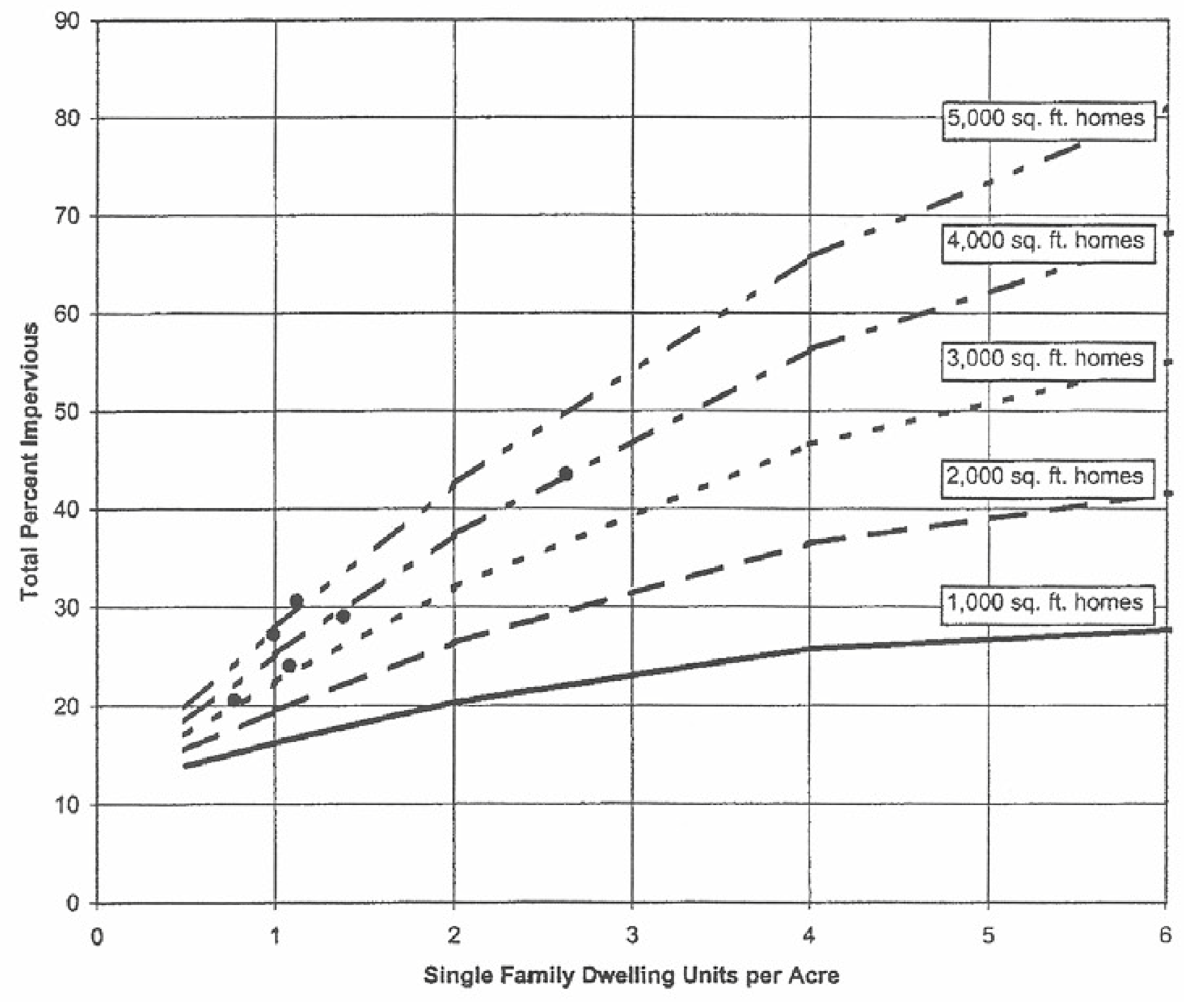


Figure 3-3
Impervious Percentage for Single-Family Residential Two-Story Homes



3.4 NATURAL RESOURCES CONSERVATION SERVICE (NRCS) METHOD

The NRCS Unit Hydrograph was derived from a large number of natural unit hydrographs for watersheds varying widely in size and geographic location. The NRCS UH Method, which uses the unit hydrograph theory as a basis for runoff computations, computes a hydrograph for a unit amount of rainfall excess applied uniformly over a sub-watershed for a given unit of time (or unit duration). The excess rainfall is then transformed to a subwatershed hydrograph by superimposing each excess hydrograph lagged by the unit duration.

There are many references for this method, but the City recognizes TR-55 “Urban Hydrology for Small Watersheds,” (SCS 1986) as the original source and the National Engineering Handbook, Part 630 (NRCS date varies) as the most recent explanation of the method. Hydrology Technical Note N4, can be consulted with regard to updates to the time of concentration Manning’s “n” values. This document can be found at: http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/WinTR55.html.

The shape of the NRCS UH is based on studies of various natural unit hydrographs. The basic governing parameters of this curvilinear hydrograph are as follows:

- The time-to-peak, T_p, of the unit hydrograph approximately equals 0.2 times the time-of-base, T_b.
- The point of inflection of the falling leg of the unit hydrograph approximately equals 1.7 times T_p.

The basic assumptions made when applying the NRCS UH Method (and all other unit hydrograph methods) are:

- The effects of all physical characteristics of a given watershed are reflected in the shape of the storm runoff hydrograph for that watershed.
- At a given point on a stream, discharge ordinates of different unit graphs of the same unit time of rainfall excess are mutually proportional to respective volumes.
- A hydrograph of storm discharge that would result from a series of bursts of excess rain or from continuous excess rain of variable intensity may be constructed from a series of overlapping unit graphs each resulting from a single increment of excess rain of unit duration.

3.4.1 Watershed Sizing

The determination of the peak rate of runoff at a given design point is affected by the number of sub-watersheds within a larger watershed. Recommended guidelines are:

- For watersheds up to 100 acres in size, the maximum sub-watershed size should be approximately 20 acres.
- For watersheds over 100 acres in size, increasingly larger sub-watersheds may be used as long as the land use and surface characteristics within each sub-watershed are relatively homogeneous and as long as the size of the watersheds within a model do not vary more than one order of magnitude. In addition, the sub-watershed sizing should be consistent with the level of detail needed to determine peak flow rates at various design points within a given watershed.

3.4.2 Lag Time

For the NRCS UH Method, the time of concentration is used to determine the time-to-peak, t_p, of the unit hydrograph and subsequently, the peak runoff. Input data for the NRCS Dimensionless UH Method (SCS 1985) consists of a single parameter, T_{LAG}, which is equal to the lag (in hours) between the center of mass of rainfall excess and the peak of the unit hydrograph. For small watersheds (less than 1 square mile), the lag time is related to the time of concentration, t_c, by the following empirical relationship:

$$T_{LAG} = 0.6\ t_c \tag{3.7}$$

Where:

T_{LAG} = time (hours) between the center of mass of the rainfall excess and the peak of the unit hydrograph.

t_c = time of concentration (min). (See Section 3.3.1; the same time of concentration method may be used as the Rational Method.)

For large watersheds (greater than 1 square mile), the lag time (and t_c) is generally governed mostly by the concentrated flow travel time, not the initial overland flow time. In addition, as the watershed gets increasingly larger, the average flow velocity (and associated travel time) becomes more difficult to estimate. Therefore, for these watersheds, Equation 3.8 is recommended for use in computing T_{LAG}:

$$T_{LAG} = 26\, K_n\, (L\, L_c / S^{½})^{1/3} \qquad (3.8)$$

Where:

K_n = A length-weighted average Manning's roughness factor for the watershed channels.

L = Length of longest watercourse (miles)

L_c = Watercourse length from the outflow point to a point on the channel nearest the centroid of the watershed (miles)

S = Average slope of the longest watercourse (ft per mile)

This lag equation is based on the United States Bureau of Reclamation's (USBR) analysis of the above parameters for several watersheds in the Southwest desert, Great Basin, and Colorado Plateau area (USBR 1989).

3.4.3 Precipitation Losses

Land surface interception, depression storage, and infiltration are referred to as precipitation losses. Interception and depression storage represent the surface storage of water by trees or grass, in local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in a surface area where water is not free to move as overland flow. Infiltration represents the movement of water through the soil beneath the land surface.

Three important factors should be noted about precipitation loss computations:

- Precipitation which does not contribute to the runoff process is considered to be lost from the system.
- The equations used to compute the losses do not provide for soil moisture or surface storage recovery (i.e., contribute to surface runoff); therefore, the calculated amount of runoff is conservatively high.
- Precipitation losses are considered to be a uniformly distributed over an entire sub-watershed.

CN Method

To estimate precipitation losses, the NRCS relates the watershed characteristics of soil groups to a curve number, CN (SCS 1985). The NRCS provides information relating soil group type to the curve number as a function of soil cover, land use type, and antecedent moisture conditions, which can be determined from the soil name. Soil data can be obtained online through the NRCS Web Soil Survey at http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm.

Precipitation loss is calculated based on supplied values of CN and I_A, which are related to a total runoff depth for a storm by the following relationships:

$$Q = (P - I_A)^2 / ([P - I_{A]} + S) \quad (3.9)$$

$$S = (1{,}000 / CN) – 10 \quad (3.10)$$

$$I_A = 0.2\ S \quad (3.11)$$

Where:

Q = Accumulated Excess (in)
P = Accumulated Rainfall Depth (in)
I_A = Initial abstraction (in)
S = Currently Available Soil Moisture Storage Deficit (in)
CN = SCS Curve number

Note that initial abstraction, I_A, (i.e., soil surface storage capacity) is based on empirical evidence established by the NRCS, and is the default value in HEC-1 Program (USACE 1988). Since the NRCS Method results in total excess for a storm, the incremental excess (the difference between rainfall and precipitation loss) for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

CN Determination

The NRCS Curve Number Method uses a soil cover curve number (CN) for computing excess precipitation. The CN is related to hydrologic soil group (A, B, C, or D), land use, treatment class (cover), and antecedent moisture condition.

The Hydrologic Soil Group (HSG) is determined from published soil maps for the area, in which each soil type correlates to either Group A, B, C, or D. The latest soil maps are available online through the NRCS Web Soil Survey (WSS) at http://websoilsurvey.nrcs.usda.gov/app. Instructions for obtaining soil data through the WSS are included on the website. For larger watersheds, the NRCS Soil Data Mart may also be used to obtain soils data.

The amount of rainfall in a period of 5 to 30 days preceding a particular storm is referred to as antecedent rainfall, and the resulting condition of the watershed in regard to potential runoff is referred to as an antecedent moisture condition. In general, higher amounts of antecedent rainfall result in greater amounts of runoff from a given storm. The effects of infiltration and evapotranspiration during the antecedent period are also important, as they may increase or lessen the effect of antecedent rainfall. Because of the difficulties of determining antecedent storm conditions from data normally available, the conditions are reduced to three cases, AMC-I, AMC-II, and AMC-III. For the City, an AMC-II condition is recommended for determining storm runoff.

3.4.4 Channel Routing

Whenever a large or a non-homogeneous watershed is being investigated, the watershed should be divided into smaller and more homogeneous sub-watersheds. The storm hydrograph for each sub-watershed is then routed through the channel and combined with individual sub-watershed hydrographs to develop a storm hydrograph for the entire watershed. Channel routing is generally not required for channels that have a travel time less than 5 minutes.

The computer software packages for hydrograph routing offer the kinematic wave method or the dynamic wave methods. These methods divide the channel reach into segments and apply the momentum or energy equations, respectively, to the channel wave to estimate the effects of energy loss in the channel. See the software documentation for detailed explanations of these procedures and the limitations of their use. HEC-HMS offers several other methods, including the Muskingum, Muskingum-Cunge, Modified Puls, and Lag channel routing methods. The introduction to Chapter 8 in the HEC-HMS technical reference manual should be consulted for use of these methods.

3.5 RESERVOIR ROUTING AND DETENTION

Storm runoff detention is required for new development and, therefore, detention reservoirs will be required. In some instances, the sizing of the detention storage will be based upon hydrograph storage routing techniques rather than direct calculation of volume and discharge requirements. See Section 10, Detention, for a discussion of these methods and their application.

3.6 STANDARD FORMS

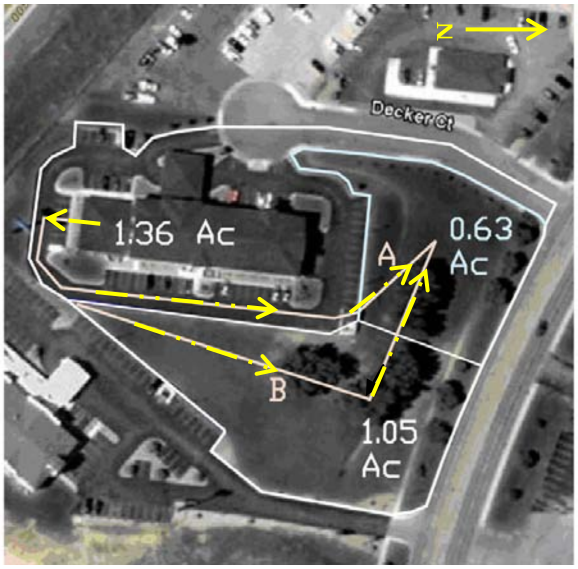
The forms SF-1 and SF-2 at the end of this Section may be used for calculation of Time of Concentration and Storm Drainage System Design using the Rational Method.

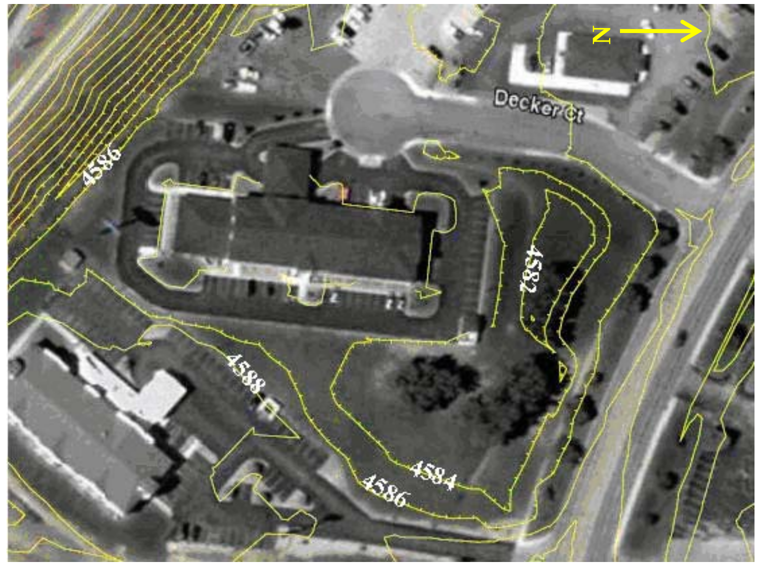
3.7 DESIGN EXAMPLE

Rational Method Example.

The following example uses the sketch in Figure 3-4.

Figure 3-4
Plans for Design Example





This example uses two approaches to estimating the peak flow at the outfall to a hotel development in Gillette, and illustrates the need to consider the runoff from just the impervious portion of the site as well as the entire site.

The examined area (Figure 3-4, right photo) does not have offsite run-on, and has been divided into 3 sub-catchments (Figure 3-4, left photo). The longer flow path (B) was combined with the whole site, while the impervious area was combined with the shorter flow path (A).

The overland flow length is assumed as the length of 50 feet from the back of the sidewalk on the far side of the hotel to the curb. The slope is measured from the terrain as 0.5% for this length, and the C_5 from Table 3-3 for 100 percent impervious is 0.9. The overland travel time t_i for the time of concentration for flow path A may be calculated using Equation 3.3 as:

$$t_i = 1.8\ (1.1 - C_5)\ L_o^{½} / S^{1/3}$$

$$\text{or } t_i = 1.8\ (1.1 - 0.9)\ 50^{½} / 0.5^{1/3} = 3.2 \text{ minutes.}$$

The travel time may be estimated using Manning's equation (see Equation 4.1). Assuming R approximately equals the depth and that the depth is about 0.2 ft:

$$Q = (1.49/n)\ AR^{2/3}\ \sqrt{S} \text{ or } V = (1.49/n)\ R^{2/3}\ \sqrt{S}$$

or $V = (1.49/.013)\ 0.2^{2/3}\ \sqrt{.005} = 2.8$ feet per second, which is a reasonable speed for gutter flow at about 0.5%. Channel travel time t_t is:

344 feet / 2.8 feet per second / 60 seconds per minute = 2.1 minutes.

Using Equation 3.2 the time of concentration is:

$t_c = t_i + t_t$ or $3.2 + 2.1 = 5.4$ minutes for flow path A.

Similarly for flow path B, Assuming 200 feet of overland flow over grass with a C = 0.3 for C and D type soils from Table 3-3, and a measured slope of 1.0%:

$$t_i = 1.8\ (1.1 - 0.3)\ 200^{½} / 1.0^{1/3} = 14.1 \text{ minutes.}$$

The travel time may be estimated using Manning's equation (see Equation 4.1). Assuming R approximately equals the depth and that the depth is about 0.2 ft:

$Q = (1.49/n)\ AR^{2/3}\ \sqrt{S}$ or $V = (1.49/n)\ R^{2/3}\ \sqrt{S}$

or $V = (1.49/.03)\ 0.2^{2/3}\ \sqrt{.01} = 1.7$ feet per second, which is a conservative speed for shallow flow in a grass swale at about 1%. Channel travel time t_t is:

270 feet / 1.7 feet per second / 60 seconds per minute = 2.7 minutes.

Using Equation 3.2 the time of concentration is:

$t_c = t_i + t_t$ or 14.1 + 2.7 = 16.8 minutes for flow path B.

Peak flow rates may then be estimated using the Rational Method formula as expressed in Equation 3.1. We will use area-weighted C values and use the customary assumption that the intensity is equal to the intensity at the time of concentration from Table 2-4.

For flow path A:

Weighted C = {(1.36 * 0.9) + (0.63 * 0.3)/1.99 = 0.59.

t_c = 5.4 minutes, so the intensity is 4.24 by rounding on Table 2-4.

The flow rate is then:

Q = CIA = 0.59*4.24*1.99 = 5.0 cfs.

For flow path B:

Weighted C = {(1.36 * 0.9) + (1.68 * 0.3)/3.04 = 0.57.

t_c = 16.8 minutes, so the intensity is 2.67 by rounding on Table 2-4.

The flow rate is then:

Q = CIA = 0.57*2.67*(1.36 + 1.68) = 4.6 cfs.

By comparing the two results above it may be seen that the smaller impervious area generates a larger calculated flow rate than the larger, partially impervious area.

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Standard Form SF-1. Time of Concentration

PROJECT ______________________________
PREPARED BY ______________________________ DATE ________________

Sub-Basin Data			Initial/Overland Time (t_i)			Travel Time (t_t)				t_c Check (Urbanized Basins)		Final t_c	Remarks
DESIGN	C_5	AREA (A) acres	LENGTH (L) feet	SLOPE (S) %	t_i minutes	LENGTH (L) feet	SLOPE (S) %	Velocity (V) ft/sec	t_t minutes	TOTAL LENGTH (L) feet	t_c=(L/180)+10 minutes	$t_c = t_i + t_t$ minutes	

Standard Form SF-2. Storm Drainage System Design (Rational Method Procedure)

CALCULATED BY:	JOB NO.:
DATE:	PROJECT:
CHECKED BY:	DESIGN STORM:

STREET		DIRECT RUNOFF							TOTAL RUNOFF				STREET		PIPE			TRAVEL TIME			REMARKS
	DESIGN POINT	AREA DESIGN	AREA (A)	RUNOFF COEFF. (C)	t_c	CA	I	Q	t_c	Σ(CA)	I	Q	SLOPE	STREET FLOW	DESIGN FLOW	SLOPE	PIPE SIZE	LENGTH	VELOCITY	t_t	
			ac		min		in/hr	cfs	min		in/hr	cfs	%	cfs	cfs	%	in	ft	ft/sec	min	

SECTION FOUR
OPEN CHANNELS

4.1 INTRODUCTION

Presented in this section are technical criteria and design standards for hydraulic evaluation and design of open channels (natural and artificial). Open-channel flow can be extremely complex. In many instances, special design or evaluation techniques will be required. Discussions and hydraulic standards are provided for different channel types that can be found or are acceptable in the City. The information presented in this section is the minimum upon which channel evaluation and design shall be based. The ultimate responsibility for design of safe and stable channels rests with the professional design engineer.

A good understanding of site conditions is vital to stable channel design. Additional analysis that goes beyond the scope of the Drainage Criteria may be necessary for unique or unusual channel conditions. For additional information, designers are encouraged to refer to textbooks and other technical publications addressing this subject.

4.2 CHANNEL TYPES, MAJOR AND MINOR DRAINAGEWAYS

Open channels can be categorized as either natural or engineered (artificial). Natural channels include all watercourses that are carved and shaped by erosion and sediment transport processes. Artificial channels are those constructed by human efforts.

4.2.1 Natural Channels

Streams, rivers, and watercourses are carved and shaped by natural erosion processes before urbanization occurs. As the tributary watershed urbanizes, the altered nature of the runoff peaks and volumes from urban development will inevitably cause erosion of natural channels, resulting in the need to provide grade-control checks and localized bank protection for stabilization.

Hydraulic investigations are necessary to assure that the natural channels will be adequate for conveyance of developed runoff.

4.2.2 Grass-Lined Channels

Among various types of constructed or modified channels, grass-lined channels are most desirable. They provide channel storage, lower velocities, groundwater recharge, and various multiple use benefits. Areas along the thalweg may need to be concrete, rock-lined, or otherwise reinforced with vegetation to minimize erosion and maintenance problems from minor flows.

4.2.3 Concrete-Lined Channels

Concrete-lined channels are typically used in confined areas. These are high velocity, artificial drainageways that are not encouraged. However, in retrofit situations where existing flooding problems need to be solved and where right-of-way is limited, concrete channels may offer advantages over other types of open channels. Special attention needs to be given to provide safety measures (e.g., fencing) around the concrete-lined channels and maintenance access.

4.2.4 Riprap-Lined Channels

Riprap-lined channels are channels in which riprap is used for lining of the channel banks and the channel bottom to control erosion. Riprap is commonly used to control erosion in both channel banks and beds, transition sections upstream and downstream of hydraulic structures, at bends, and bridges. Loose or grouted riprap is cost effective for short channel reaches. Riprap lining might also be appropriate for:

1. Flows that produce channel velocities in excess of allowable values
2. Channels where side slopes need to be steeper than 3H:1V
3. Channel bends and transitions
4. Low-flow channels

Riprap-lined channels offer a compromise between a grass-lined channel and a concrete-lined channel. They can reduce right-of-way needs as compared to grass-lined channels and be less expensive than concrete-lined channels. However, safety and maintenance access considerations are similar to those of concrete-lined channels.

4.2.5 Wetlands-Vegetation-Bottom Channels

Wetland-bottom channels are similar to grass-lined channels, except that wetland vegetation growth is encouraged by eliminating the concrete-lined trickle channel and flattening longitudinal slope so that low flows have low velocities. Under certain circumstances, such as when existing wetland areas are affected or natural channels are modified, a Corps of Engineers Section 404 permit might require the use of channels with wetland vegetation, or a wetland-bottom channel may better suit individual site needs to mitigate wetland impacts or to enhance urban stormwater quality.

Wetlands-vegetation-bottom channels offer other benefits including wildlife habitat, groundwater recharge and water quality enhancement. In low-flow areas, the banks may need supplemental reinforcement to protect against undermining. Generally, channels that carry a small base flow on a flat gradient are better suited for wetland-bottom channels.

4.2.6 Other Lining Types

A variety of artificial channel liners are on the market, all intended to protect the channel walls and bottom from erosion at higher velocities. These include gabions, interlocking concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fiber liners. As with rock and concrete liners, all of these types are best considered for helping to solve existing urban flooding problems and are not recommended for new developments. Each type of liner has to be evaluated for its merits, applicability, and cost effectiveness, how it meets other community needs, long term integrity, and maintenance needs and costs. Generally, channels lined with artificial materials are permitted on a case-by-case basis in new development areas of the City.

4.2.7 Selection of Channel Type

For all open channels, hydraulic analyses must be conducted to evaluate flow characteristics, including flow regime, water surface elevations, velocities, depths, and hydraulic transitions for multiple flow conditions. Each type of channel must be evaluated for its longevity, integrity, maintenance requirements and costs, and general suitability for community needs, among other factors. Wherever practical, open channels shall have mild flow characteristics, be wide and shallow, be compatible with surrounding land uses, and be "natural" in appearance. Open channels should be designed to integrate recreation and aesthetic needs, protect wildlife, and support natural plant populations. Selection of the most appropriate channel type for the conditions that exist at a project site shall be based on a multi-disciplinary evaluation of the following factors:

- **Hydraulics** – Slope of thalweg, right-of-way, conveyance, basin sediment yield, topography, ability to drain adjacent lands.
- **Structural –** Cost, availability of material, areas for wasting excess excavated material, seepage and uplift forces, shear stresses, pressures and pressure fluctuations, momentum transfer.
- **Environmental** – Neighborhood character and aesthetic requirements, need for new green areas, street and traffic patterns, municipal or county policies, wetland mitigation, wildlife habitat, water quality enhancement.
- **Sociological –** Neighborhood social patterns, children, pedestrian traffic, and recreational needs.
- **Maintenance –** Design life, repair and reconstruction needs, proven performance, and accessibility.
- **Regulatory –** Federal floodplain and environmental regulations, and state and local water quality regulations.

4.3 TYPES OF OPEN-CHANNEL FLOW AND COMPUTATIONS

The basic equations and computational procedures for uniform, gradually varying, and rapidly varying flow are presented in the following paragraphs. The designer is encouraged to review the many textbooks written on this subject for detailed discussions of the technical procedures.

4.3.1 Uniform Flow

Open-channel flow is uniform if the flow is steady and the depth of flow and velocity are the same at every section of the channel. For a given channel geometry, roughness, discharge, and slope, there is only one possible depth for maintaining uniform flow, referred to as the "normal depth." The computation of uniform flow and normal depth is based upon Manning's Equation:

$$Q = (1.49/n)\, AR^{2/3} \sqrt{S} \quad (4.1)$$

Where:

Q = flow rate (ft^3/sec)
n = Manning roughness coefficient

A = area (ft^2)
P = wetted perimeter (ft)
R = hydraulic radius, R = A/P (ft)
S = slope of the energy grade line (ft/ft)

For trapezoidal channels, the EGL, HGL, and the channel bottom can be assumed parallel for uniform, normal depth flow conditions. Table 4-1 lists Manning's roughness coefficient (n) values for many types of open-channel surfaces.

Table 4-1
Typical Roughness Coefficients (Manning's n) for Open Channels

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
A. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
B. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
C. Excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
D. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
E. Channels not maintained, weeds and brush			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same as above, but highest state of flow	0.045	0.070	0.110
4. Dense brush, high state	0.080	0.100	0.140

Table 4-1
Typical Roughness Coefficients (Manning's n) for Open Channels

Type of Channel and Description	Minimum	Normal	Maximum
LINED OR BUILT-UP			
A. Concrete			
1. Straight culvert, free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and debris	0.011	0.013	0.014
3. Straight drain with manholes, inlet, etc.	0.013	0.015	0.017
4. Trowel Finish	0.011	0.013	0.015
5. Float Finish	0.013	0.015	0.016
6. Unfinished, steel form	0.012	0.013	0.014
7. Unfinished, smooth wood form	0.012	0.014	0.016
8. Unfinished, rough wood form	0.015	0.017	0.020
9. Gunite, good section	0.016	0.019	0.023
10. Gunite, wavy section	0.018	0.022	0.023
B. Concrete Bottom (Float Finish) with Sides of:			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Riprap or dry rubble	0.020	0.030	0.035
C. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
D. Gravel Bottom with Sides of:			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Riprap or dry rubble	0.023	0.033	0.036
E. Asphalt			
1. Smooth	0.013	0.013	--
2. Rough	0.016	0.016	--

Source: Chow, V.T., "Open Channel Hydraulics", 1959.

4.3.2 Gradually Varying Flow

The most common occurrence of gradually varying flow in storm drainage is the backwater created by culverts, storm drain inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel, and the water surface profile (a.k.a. "backwater curve") is computed using either the Direct-Step or Standard-Step Method.

The Direct-Step Method is best suited to the analysis of open prismatic channels. Water surface profiles in simple prismatic channels can be computed manually. Chow (1959) presents the basic method for applying the direct-step analysis. The Direct-Step Method is also available in many handheld and personal computer programs, such as the U.S. Army Corps of Engineers' *HEC-RAS River Analysis System*. The design engineer may use this program or proprietary computer software to compute water surface profiles for channel and floodplain analyses.

The Standard-Step Method is required for the analysis of irregular or non-uniform cross-sections. Because the Standard-Step Method involves a tedious iterative process, design engineers commonly use computer programs such as HEC-RAS to accomplish these calculations.

4.3.3 Rapidly Varying Flow

Rapidly varying flow is characterized by very pronounced curvature of the water surface profile. The change in water surface profile may become so abrupt to result in a state of high turbulence. The most common occurrence of rapidly varying flow in storm drainage applications involves weirs, orifices, hydraulic jumps, non-prismatic channel sections (transitions, culverts and bridges), and nonlinear channel alignments (bends). Each of these flow conditions requires detailed calculations to properly identify the flow capacities and depths of flow in the given section.

The design engineer must be cognizant of the design requirements for rapidly varying flow conditions, and include all necessary calculations as part of the design submittal documents. The design engineer is referred to Chow's *Open Channel Hydraulics* (1959) for the proper methods to use to design drainage facilities with rapidly varying flow characteristics.

4.3.4 Critical Flow Computation

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

1. The specific energy ($E=y + v^2/2g$) is at a minimum for a given discharge (Q).
2. The discharge (Q) is a maximum for a given specific energy (E).
3. The specific force is a minimum for a given discharge (Q).
4. The velocity head ($v^2/2g$) is equal to half the hydraulic depth (D/2) in a channel of small slope.
5. Fr is equal to 1.0.

Open channels must not be designed to flow at or near critical state ($0.80 < Fr < 1.2$). If the critical state of uniform flow exists throughout an entire channel reach, the flow is critical and the channel slope is at critical slope (Sc). A slope less than Sc results in subcritical flow. A slope steeper than Sc will cause supercritical flow. A flow at or near the critical state is unstable. Factors creating minor changes in specific energy, such as channel debris or minor variation in roughness, will cause a major change in depth.

The criteria of minimum specific energy for critical flow results in the definition of the Fr as follows:

$$Fr = v / \sqrt{gD} \tag{4.2}$$

Where:

Fr = Froude Number (dimensionless)
v = Velocity (ft/sec)
g = Gravitational acceleration (32.2 ft/sec^2)
A = Channel flow area (ft^2)
T = Top width of flow area (ft)
Dh = Hydraulic depth, $Dh = A/T$ (ft)

4.3.5 Design Procedures for Open Channels with Subcritical Flow

All open channels shall be designed in accordance with Sections 4.4, Design Criteria for Open Channels, and Section 4.5, Channel Appurtenances. The following procedures shall be used to design open channels flowing in a subcritical condition ($Fr < 0.8$).

Transitions

Subcritical transitions occur when one subcritical channel section changes to another subcritical channel section (expansion or contraction), when there is a change in longitudinal slope, or at the confluence of two or more open-channel sections. Properly designed flow transitions mimic the expansion or contraction of natural flow boundaries and minimize surface disturbances from crosswaves and turbulence. Figure 4-1 presents a number of typical subcritical transition sections.

Drop structures and hydraulic jumps are special transitions where excess energy is dissipated by design. Hydraulic jumps should be designed to occur only within energy dissipation or drop structures, and not within an erodible channel. Special transitions such as drop structures and hydraulic jumps shall meet the criteria described in Section 4.5, Channel Appurtenances.

Transition Length

The length of the transition section should be long enough to keep the streamlines smooth and nearly parallel throughout the expanding (contracting) section. Guidelines based on experimental data suggest the minimum length of the transition section is as follows:

$$L_t \geq 0.5L_c\,(\Delta T_w) \tag{4.3}$$

Where:

L_t = minimum transition length (ft)
L_c = length coefficient (dimensionless)
ΔT_w = difference in the top width of the normal water surface upstream and downstream of the transition (ft)

For approach flow velocities of 12 fps or less, the transition length coefficient (L_c) shall be 4.5. This represents a 4.5L:1W expansion or contraction, or about a 12.5-degree divergence from the channel centerline. For flow approach velocities of more than 12 ft/sec, the transition length coefficient (L_c) shall be 10. This represents a 10L:1W expansion or contraction, or about a 5.75-

degree divergence from the channel centerline. These transition length guidelines are not applicable to cylinder-quadrant or square-ended transitions.

Expansions and Contractions

The energy loss created by an expanding or contracting section may be calculated using the following equation:

$$H_t = (K_{te} \text{ or } K_{tc})(v_2^2/2g - v_1^2/2g) \quad (4.4)$$

Where:

H_t = Energy loss (ft);
K_{te} = Expansion transition coefficient
K_{tc} = Contraction transition coefficient
v_1 = Upstream velocity (ft/sec)
v_2 = Downstream velocity (ft/sec)
g = Gravitational acceleration (32.2 ft/sec^2)

Expansion and contraction loss coefficients (K_{te} and K_{tc}) values for the typical open-channel transition sections are shown in Figure 4-1.

4.3.6 Design Procedures for Superelevation

Superelevation is the transverse rise in water surface that occurs around a channel bend, measured between the theoretical water surface at the centerline of a channel and the water surface elevation on the outside of the bend. Superelevation in bends shall be estimated from the following equation:

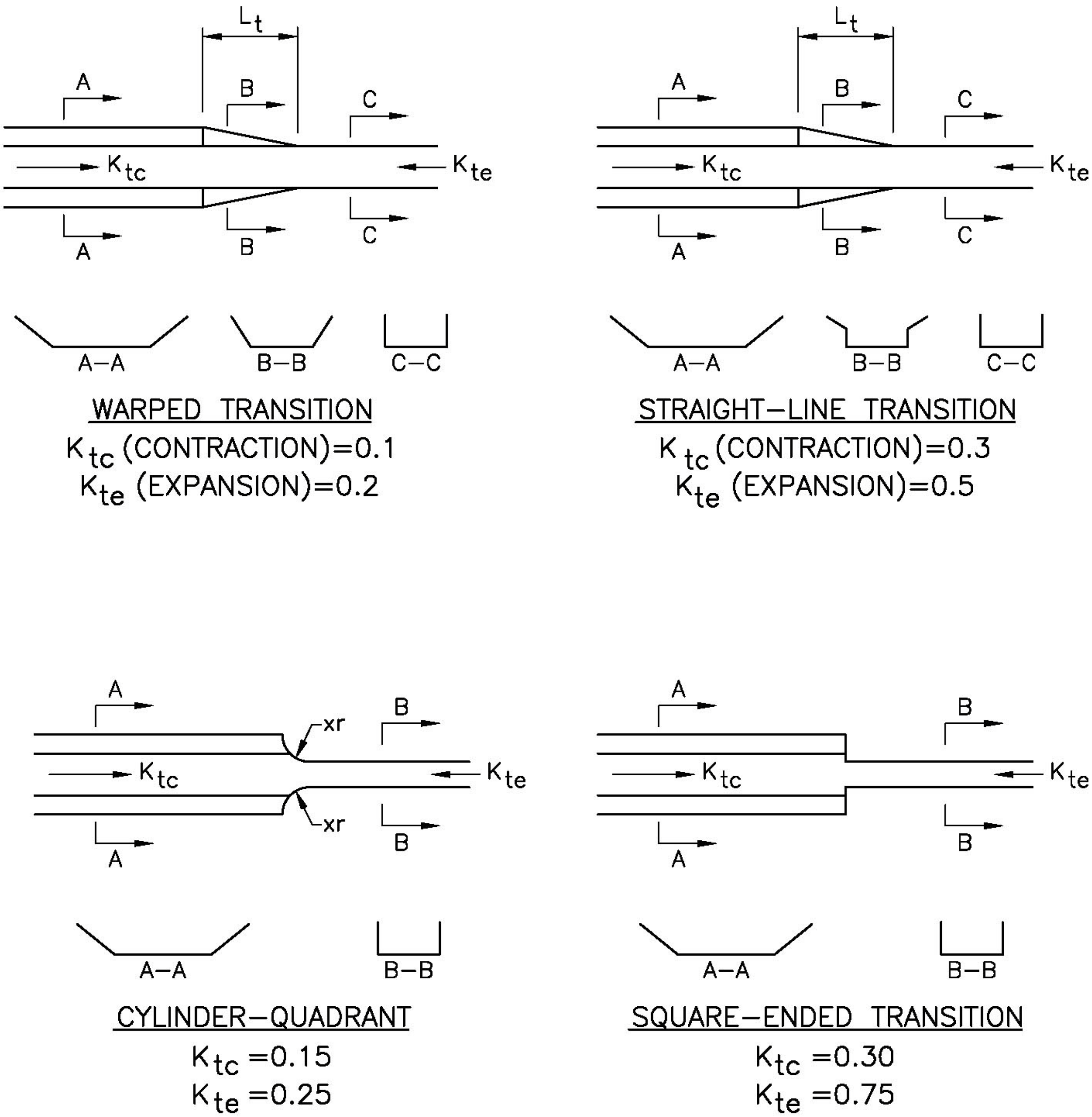
$$\Delta y = Cv^2W/rg \quad (4.5)$$

Where:

r = Radius of curvature at centerline of channel (ft)
C = Curvature coefficient, 0.5 for subcritical flow conditions
Δy = Rise in water surface between design water surface at centerline of channel and outside water surface elevation (ft)
W = Top width at the design water surface at channel centerline (ft)
v = Mean channel velocity (ft/sec)
g = Gravitational acceleration (32.2 ft/sec^2)

Bends in supercritical channels create cross-waves and superelevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, best design practice is to design the channel radius of curvature to limit the superelevation of the water surface to 1 ft or less. This can be accomplished by modifying the equation above to determine the allowable radius of curvature of a channel for a given superelevation value.

Figure 4-1
Typical Subcritical Transition Sections and Loss Coefficients



4.4 DESIGN CRITERIA FOR OPEN CHANNELS

All new open channels shall be designed to safely confine and convey the runoff from the Major event. Open channels shall be designed for a subcritical flow condition with a Froude Number no greater than 0.8. Open channels shall be designed to maintain stable cross-sections and slopes based upon the design flows to be conveyed, soil types, and channel cover conditions.

In addition, all stormwater runoff shall be accommodated so that flows resulting from all storm events, up to and including the Major event, will be safely passed to receiving channels without causing inundation of private property or any buildings that may sustain damage from inundation. Consideration should also be given to potential damage that may be caused by storms greater than the 100-year event.

Depending on the local conditions, the specific requirements for a particular type channel may be more stringent than the general criteria outlined in this section. In addition, unique and unusual site conditions may require additional design analysis be performed to verify the suitability of the proposed channel design for the project site.

4.4.1 Manning's Roughness Coefficient

Selection of an appropriate roughness coefficient for a given channel section is important for the hydraulic capacity analysis and design of open channels. The roughness coefficient can vary significantly depending on the channel type and configuration, density and type of vegetation, depth of flows, and other hydraulic properties. The design should be evaluated using a range of values to address various design parameters, higher values for depth of flow calculations, and lower values for velocity calculations.

Table 4-1 shows recommended values for the Manning's roughness coefficient for various channel types and conditions. Manning's roughness coefficients for riprap channels shall be computed based on the criteria outlined in Section 4.4.9, Riprap-Lined Channels.

4.4.2 Vertical and Horizontal Alignment

Open channels shall have a minimum longitudinal gradient of 0.5 percent whenever practical. Flatter grades may be approved with prior consultation with the City. Open channels with grades flatter than 0.5 percent shall have provisions for the drainage of nuisance low flows.

Horizontal alignment changes of two degrees or less may be accomplished without the use of a circular curve for subcritical flow designs ($Fr < 0.8$). All open channels shall have a curvilinear alignment with a centerline radius equal to two times the top width of the design flow or 100 feet, whichever is greater. Curved channel alignments shall have superelevated banks in accordance with Section 4.3.6, Design Procedures for Superelevation.

4.4.3 Minimum and Maximum Permissible Velocities

Open-channel sections shall be designed to remain stable at the design flow rate and velocity. The design flow may not always have the highest velocity; therefore, best practice is to confirm channel section stability during events smaller than the design flow. This may be accomplished by evaluating flows of specific more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions (e.g., one-half, one-quarter, and further if necessary) of the design flow. Calculations for the full design flow and the flow rate that produces the maximum velocity shall be submitted for review.

Design of open channels shall be governed by maximum permissible velocity. This method assumes that a given channel section will remain stable up to a maximum permissible velocity. Table 4-2 lists the maximum permissible velocities for the Major storm for several types of channels. Except for wetland bottom channels, the minimum velocity for the Minor storm for all open channels shall be 2 ft/sec.

Additional geotechnical and geomorphologic investigation and analyses may be required for natural channels or improved unlined channels to verify that the channel will remain stable based on maximum permissible design velocities.

Table 4-2
Maximum Permissible Mean Channel Velocity for Major Storm

Material/Lining[1]	Max Velocity (ft/s)
NATURAL AND IMPROVED UNLINED CHANNELS	
Erosive soils (HSG A & B)	3.0
Less Erosive soils (HSG C & D)	5.0
FULLY LINED CHANNELS	
Unreinforced Vegetation	
Erosive soils (HSG A & B)	5.0
Less Erosive soils (HSG C & D)	7.0
Loose Riprap	
Angular Rock	12.0
Semi-Angular Rock	12.0
Grouted Riprap	12.0
Gabions	15.0
Soil Cement	15.0
Concrete	18.0

[1] For composite lined channels, use the lowest of the maximum mean velocities used in the composite lining.

4.4.4 Freeboard

In the context of these Drainage Criteria, freeboard is the additional height of a flood control facility (e.g., channel, levee, or embankment) measured from the design water surface elevation to provide a factor of safety for wave action and overtopping.

Open-channel facilities conveying a design flow of less than 10 cfs shall have a minimum freeboard of 0.5 feet. Open-channel facilities conveying a design flow of 10 cfs or more shall have a design freeboard based on a minimum freeboard of 1-foot, with allowances for velocity, superelevation, and other water surface disturbances. The equation below describes the minimum design freeboard for subcritical flow.

$$h_{fr} = \max\left(1.0,\ 0.5 + v^2/2g + Cv^2W/rg + \Delta y\right) \tag{4.6}$$

Where:

h_{fr} = Minimum required freeboard (ft)
v = Flow velocity (ft/sec)
g = Gravitational acceleration (32.2 ft/sec^2)
Cv^2W/rg = Superelevation allowance (ft) (See Section 4.3.6)
Δy = Allowances for other hydraulic phenomenon (ft), (e.g., high debris)

A superelevation allowance shall be applied to both banks of the channel at channel bends in the following manner:

1. Begin at a point 5.0 times the characteristic wave length of the design flow ($5L_w$), measured from the downstream tangent point of the curve, with no superelevation allowance. The characteristic wavelength, L_w, shall be two times the channel top width, W.
2. Taper uniformly to the full superelevation allowance at a point 3.0 times the characteristic wave length of the design flow ($3.0L_w$), measured from the downstream tangent point of the curve.
3. Maintain the full superelevation allowance through the curve.
4. Continue the top-of-bank elevation level from the upstream tangent point of the curve to its intersection with the normal top of bank.

The freeboard under the lowest chord of a bridge deck (i.e., the soffit elevation) shall be a minimum of 1 foot during the 100-year design event. In cases where the bridge has been designed to withstand hydraulic forces of floodwaters and impact from large floating debris, the water surface elevation upstream of the bridge shall maintain a freeboard of at least 1 foot below the roadway crest and the finished floors of structures within the zone influenced by the bridge headwater.

4.4.5 Access, Safety, and Maintenance

Any easement encompassing a channel shall be wide enough to provide for the channel structure and adequate maintenance access. Easements shall be placed on one side of lot or ownership lines in new developments and in existing developments where conditions permit.

- The minimum width of any channel easement shall be the top width of channel plus 4 feet on each side of channel or 20 feet; whichever is greater.
- Channels with a top width of less than 40 feet require a minimum 12-foot wide service road parallel to one side of the channel and a 4-foot wide access on the opposite side, whenever practicable.
- Channels 40 feet or more in top width require service roads on both sides of the channel that are a minimum of 12 feet wide, whenever practicable.

In all cases, vehicular access to the channel facility must be provided at intervals of 1,000 feet or less, whenever practical. Access easements must be at least 20 feet wide, with a maximum grade of 10 percent. Access ramps shall slope down-gradient, whenever practicable.

Specific safety requirements shall be determined on a case-by-case basis in consultation with the City. As a minimum, guardrails or other approved traffic barriers shall be provided when a channel is located next to traffic, according to the American Association of State Highway and Transportation Officials (AASHTO) guidance.

Fencing or access barriers are required for channels abutting residential developments, schools, parks, and pedestrian walkways as follows:

- Fencing is required for all concrete-lined or riprap-lined channels where the design frequency storm produces a velocity that exceeds 5 feet per second or 2 feet in depth.
- Fencing or access barriers are required for all unlined natural, grass-lined, and wetland-bottom channels with side slopes steeper than 4H:1V where the design frequency storm produces a velocity that exceeds 5 feet per second or 2 feet in depth.

Gates shall be provided for maintenance and emergency access at regular intervals, with 20-foot wide gates placed 1,000 feet on center and 4-foot gates placed 500 feet on center or portion thereof. Fencing or access barriers shall be located at a minimum of 6 inches inside the easement boundary lines unless otherwise approved.

Where failure of an open-channel facility might cause flooding of a public road or structure, the facility shall have an operation and maintenance plan. These operation and maintenance plans shall specify regular inspection and maintenance at specific time intervals (e.g., annually before the wet season) and/or maintenance “indicators” when maintenance will be triggered (e.g., vegetation more than 6 inches in height). Operation and maintenance plans shall ensure that vegetation is removed or maintained on a regular basis to maintain the function of the facility.

Flood control channels require lifetime maintenance. The project owner and design engineer shall consult with the City to determine which maintenance mechanism is required for a particular project. Privately owned and maintained detention facilities shall have a recorded agreement with a covenant binding on successors or another mechanism acceptable to the City.

4.4.6 Natural Channels

Natural open channels are important drainage elements that contribute to the image and livability of an urban environment. The areas around open channels may have other uses that facilitate trails, open space, and wildlife habitat. Typical open-channel design sections for natural channels are presented in Figure 4-2.

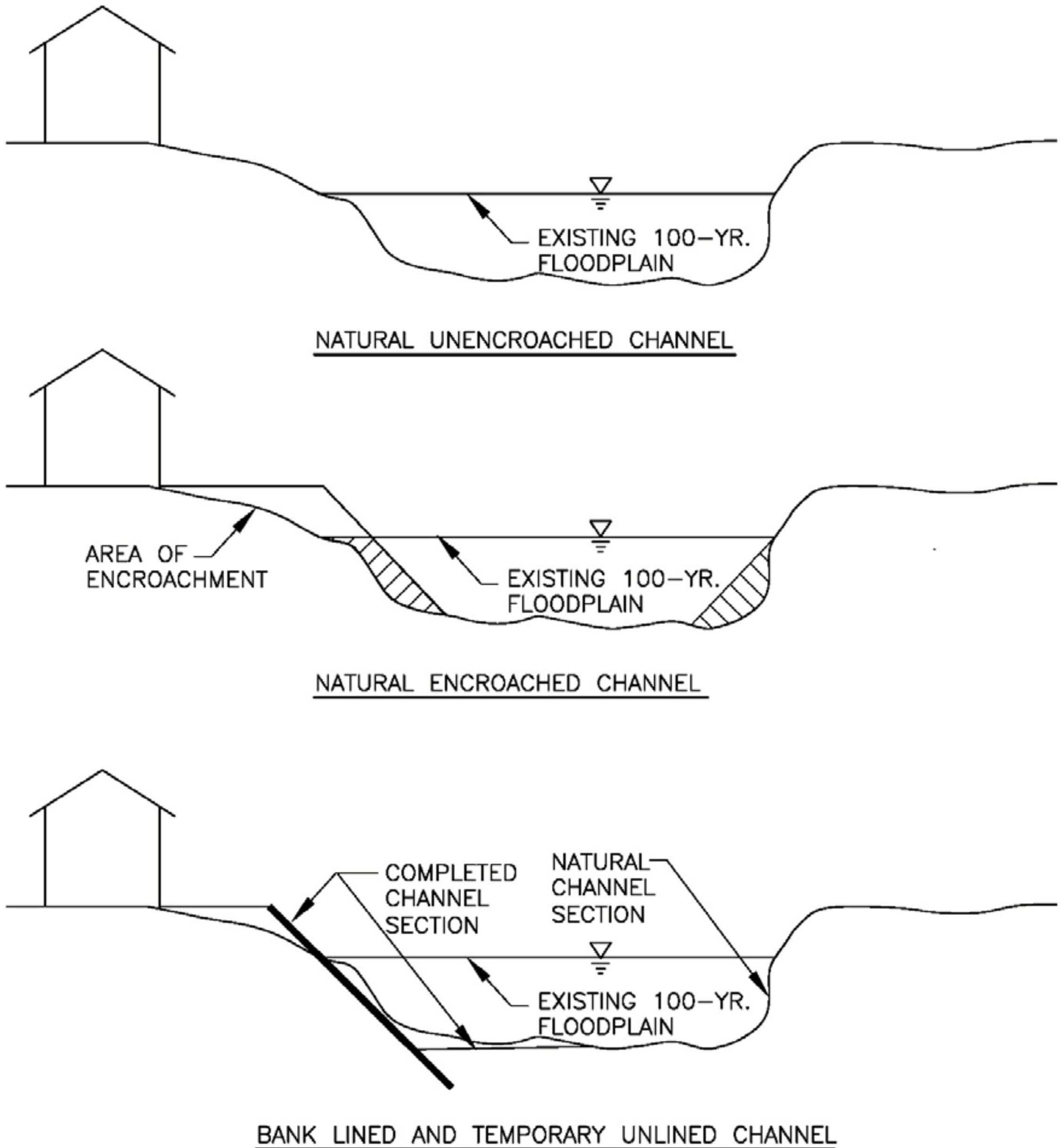
Following are design criteria for Natural Channels:

1. The channel and overbank areas shall have conveyance capacity for the 100-year flood.
2. Natural channel segments with a calculated flow velocity greater than the allowable flow velocity shall be analyzed for erosion potential. Erosion-control structures, such as drop or grade-control structures, shall be provided. Additional erosion protection may be required.
3. The water surface profiles shall be defined so that the 100-year floodplain can be delineated and protected.
4. Filling of the floodplain fringe (the area of the floodplain outside of a regulatory floodway) shall be avoided, because it reduces valuable storage capacity and may increase downstream runoff peaks.
5. Erosion-control structures, such as drop structures or check dams, will typically be required to decrease the thalweg slope and to control flow velocities, and to control erosion and sediment deposition for both the major and minor storm runoff. The appearance of these structures should be compatible with their surroundings. Where possible, structures should be located at principal grade changes to minimize the cost of retaining structures, reduce perceived scale and appearance of mass and bulk, and use

existing land forms of the site. All check drops, dams, or structures should, whenever feasible, use natural materials to integrate with natural landscape characteristics and should be provided only where necessary as indicated by hydraulic analyses.

6. Plan and profile information (i.e., HEC-2 or HEC-RAS output) for both existing and proposed floodplain and floodway conditions upstream, downstream and through the site shall be prepared and provided in the drainage report.
7. The engineer shall verify, through stable channel (normal depth) calculations, the suitability of the floodplain to contain the flows. If this analysis demonstrates erosion outside of the designated flow path (easement and/or ROW), an analysis of the equilibrium slope and degradation or aggregation depths is required and suitable improvements identified.
8. Plan and profile drawings of the floodplain shall be prepared. Appropriate allowances for known future roadway crossing or other structures, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis. The applicant shall contact the Engineering Department for information on future bridges and roads in undeveloped areas.
9. Natural waterway channel boundaries and alignments shall be preserved, maintained, or enhanced in their natural condition to serve as landscape and visual amenities, to provide focal points for development projects. To help define "edges" in and around communities, vegetation groups, rock outcroppings, terrain form, soils, waterways, and bodies of water shall be preserved to the extent practicable.
10. The usual rules of freeboard depth, curvature, and other factors, which are applicable to artificial channels, do not apply for natural channels. A minimum of 1 foot of freeboard above the 100-year water surface shall be provided, with 3 feet provided at bridges and 18 inches at structures. Significant benefits may be realized if channel overtopping and localized flooding of adjacent areas are planned for the major runoff peak.
11. If a natural channel is to be used as a major drainageway for a development, then the applicant shall meet with the Engineering Department to discuss the concept and obtain the requirements for planning and design documentation. Approval of the concept and design will be made in accordance with the requirements of Section 101 of the Gillette Design Standards.

Figure 4-2
Typical Natural Open-Channel Design Sections



4.4.7 Grass-Lined Channels

The City prefers grass-lined channels for new development when conditions allow. Grass-lined channels shall be designed in accordance with the following criteria and any special considerations to address site-specific requirements:

Channel Geometry

1. Grass-lined channels shall generally have longitudinal slopes between 0.5 percent and 0.8 percent, but are ultimately determined by the maximum permissible velocity requirements.
2. Side slopes of a grass-lined channel shall not be steeper than 4(H):1(V).
3. The channel bottom width selected shall be based on factors such as possible wetland mitigation requirements, constructability, channel stability, maintenance requirements, and multi-use purposes. The minimum channel bottom width shall be 5 feet for channels with a concrete trickle channel, 20 feet in channels with a riprap trickle channel, and 30 feet in channels with a low-flow channel.
4. The main channel depth of flow shall be no greater than 5 feet, measured from top of the trickle channel edge treatment or from the top of the low-flow channel bank, for a 100-year flow of 1,500 cfs or less.
5. The center line curvature of an open channel with subcritical flow shall have a radius of twice the surface spread of the 100-year flow, unless erosion protection is provided. The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

Horizontal Alignment and Bend Protection

The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend; therefore, it is often necessary to provide supplemental erosion protection at bends in grass-lined channels.

No channel bend protection is required along bends where the radius is greater than 2.0 times the top width of the 100-year water surface. When erosion protection is provided, the minimum radius of curvature shall be 1.2 times the 100-year flow surface spread, or 50 feet, whichever is greater. Erosion protection shall extend downstream from the end of the bend a distance equal to the length of the bend measured along the channel centerline.

Trickle and Low-Flow Channels

In urbanized areas, lawn irrigation and runoff from directly connected impervious areas contribute to continuous flows in drainageways. Therefore, a trickle channel is required on all grass-lined channels in urbanized areas with a 100-year design flow greater than or equal to 75 cfs. Trickle channels shall convey 5 percent of the 100-year design flow rate or 5 cfs, whichever is greater. Trickle channels shall have a minimum bottom width of 5 feet, and a minimum longitudinal slope of 0.5 percent. There is no freeboard requirement for trickle channels. Care shall be taken to ensure that low flows enter the trickle channel without flow paralleling the trickle channel or bypassing inlets to the channel.

Because of low maintenance and limited infiltration, concrete trickle channels are preferred. Concrete trickle channels help minimize erosion, silting, and excessive plant growth. Figure 4-3 illustrates typical concrete trickle channel sections. The concrete trickle channel shall have a minimum depth of 3 inches if a valley section is used and 6 inches if a rectangular section is used. A Manning's roughness coefficient value of n = 0.015 shall be used to design the concrete trickle channel.

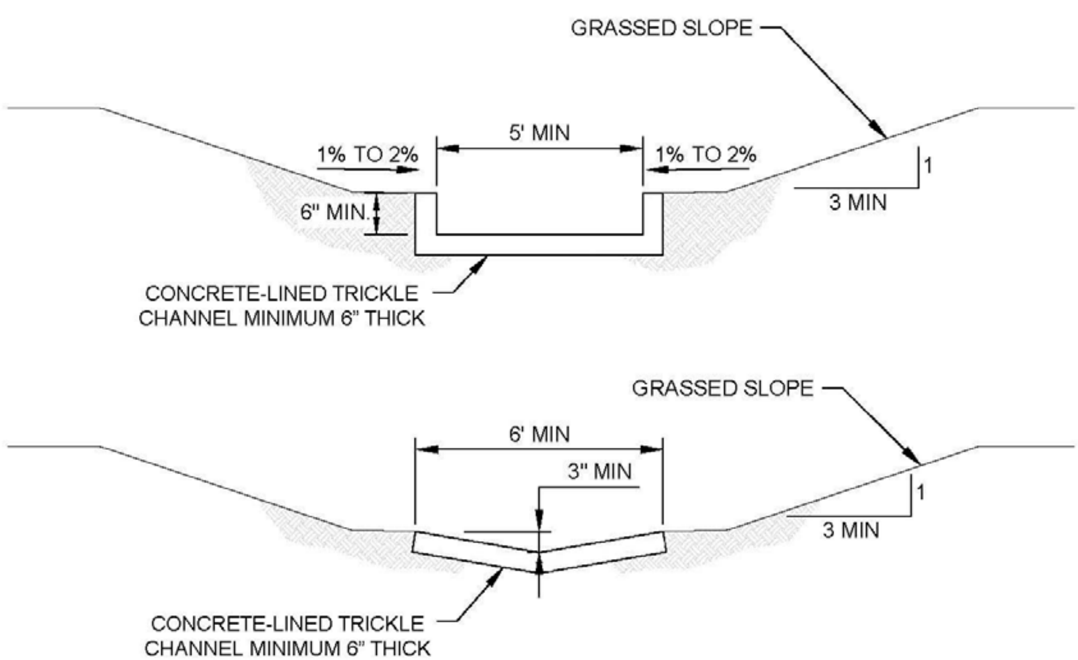
For larger streams a low-flow channel may be more appropriate than a trickle channel. Low-flow channels are used to contain relatively frequent flows within a recognizable channel section. Low-flow channels are recommended for channels with a 100-year flow greater than 200 cfs, and have capacity to convey the 2-year flow event with no freeboard. The overall flow capacity of the main channel shall include the capacity of the low-flow channel. Low-flow channels shall have a minimum depth of 12 inches. The maximum side slopes of the low-flow channel shall be 1(H):1(V). A typical low flow channel section is shown on Drawing 02725-04 in the Gillette Design Standards.

Trickle channels are not required for swales and grass-lined channels conveying a 100-year peak runoff of 75 cfs or less when adequate longitudinal slope is provided.

Vegetation and Maintenance

Satisfactory performance of a grass-lined channel depends on constructing the channel with the proper shape and preparing the area in a manner to provide conditions favorable to vegetative growth. Between the time of seeding and the actual establishment of the grass, the channel is unprotected and subject to considerable damage unless interim erosion protection is provided. See Section 11, Erosion and Sediment Control, for requirements.

Figure 4-3
Typical Cross-Section of Concrete-Lined Trickle Channel



4.4.8 Concrete-Lined Channels

Where concrete-lined channels are proposed due to ROW or other restrictions that prohibit grass-lined channels, they shall be designed in accordance with the general open channel criteria, any specific considerations due to project-specific site requirements, and the following:

Channel Geometry

1. The longitudinal slope for concrete-lined channels shall be set to limit the design velocity to 18 fps or less. Concrete-lined channels have the ability to accommodate supercritical flow conditions.
2. The minimum bottom width of the concrete-lined channel shall be 5 feet. The bottom shall be sloped to confine the low flows to the middle or one side of the channel. Side slopes shall be no steeper than 1.5(H):1(V), but vertical walls are allowed. Proper maintenance access must be provided.
3. Appropriate Manning's roughness coefficients for concrete-lined channels are listed in Table 4-1. For subcritical flow in concrete-lined channels, check the Froude Number using a Manning's roughness coefficient of n=0.011.
4. Freeboard shall be provided according to equation 4.6 with a 2-foot minimum.
5. If a retaining wall structure is used, the structure shall be designed by a structural engineer registered in the State of Wyoming and structural design calculations shall be submitted for review and approval.

Concrete Lining

1. Concrete-lining for channels shall have a minimum thickness of 6 inches and be reinforced.
2. Channels shall be continuously reinforced with transverse joints. Expansion/contraction joints (without continuous reinforcement) shall be installed only where the new concrete lining is connected to a rigid structure or to an existing concrete lining that is not continuously reinforced. The design of the expansion joint shall be coordinated with the City.
3. Longitudinal joints, where required, shall be constructed on the sidewalls at least 1 foot vertically above the channel invert.
4. All joints shall be designed to prevent differential movement.
5. Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint and the concrete lining shall be thickened at the joint.
6. The surface of the concrete lining shall be provided with a wood float finish unless the design requires additional finishing treatment. Excessive working or wetting of the finish shall be avoided if additional finishing is required.
7. A transverse concrete cutoff shall be installed at the beginning and end of the concrete-lined section of channel and at a maximum spacing of 90 feet. The concrete cutoffs shall

extend 3 feet below the bottom of the concrete slab and across the entire width of the channel lining.

Subgrade

A Geotechnical report shall be submitted that addresses the bedding and under drain requirements for the specific concrete section under consideration.

4.4.9 Riprap-Lined Channels

Where permitted, riprap-lined channels shall be designed in accordance with the following criteria and any special considerations arising out of site-specific situations. These standards for riprap-lined channels shall also be used for open channel transitions and bends to be lined with riprap. Figure 4-4 provides typical cross-sections for riprap-lined channels.

Channel Geometry

1. The longitudinal slope for riprap-lined channels shall be set to limit the design velocity to 12 fps or less. In steeper terrain, drop structures may be used to achieve the desired design velocities.
2. The channel bottom width shall be set considering constructability, channel stability and maintenance, multi-use purpose, and trickle/low-flow channel requirements.
3. The design depth of flow for the Major storm shall not exceed 7 feet in areas of the channel cross-section outside the low-flow or trickle channel.
4. Maximum side slopes shall be 2.5(H):1(V) for riprap-lined channels.
5. Freeboard shall be provided according to equation 4.6 with a 2-foot minimum.

Roughness Coefficients

The Manning's roughness coefficient for hydraulic computations may be estimated for loose riprap using Equation 4.7.

$$n = 0.0395\,(d_{50})^{1/6} \qquad (4.7)$$

Where:

d_{50} = mean stone size (ft)

This equation (Anderson 1968) does not apply to grouted riprap (n = 0.023-0.030) or to very shallow flow (hydraulic radius is less than or equal to 2.0 times the maximum rock size) where the roughness coefficient will be greater than indicated by the formula.

Toe Protection and End Treatments

Where only channel banks are to be lined, additional riprap is necessary for long-term stability. In this case:

1. The riprap blanket shall extend a minimum of 3 feet below the proposed channel bed, and the thickness of the blanket below the proposed channel bed shall be increased to a minimum of 3.0 times d_{50} to accommodate possible channel scour during floods. If the

velocity exceeds the permissible velocity requirements of the soil comprising the channel bottom, a scour analysis shall be performed to determine whether the toe requires additional protection.

2. At the upstream and downstream termination of a riprap lining, the thickness shall be increased 50 percent for at least 3 feet to prevent undercutting. Depending on the site-specific conditions, concrete cutoff walls at both ends may be necessary.
3. Transverse concrete cutoff walls may be required for riprap-lined channels where a resulting failure of the riprap lining may seriously affect the health and safety of the public. The designer shall consult with the City prior to design of riprap-lined channels to determine whether concrete cutoff walls are required and requirements for their sizing and spacing.
4. For locations where scour potential is amplified by turbulence due to rapid changes in channel geometry, such as transitions and bridges, the riprap lining thickness shall be increased by one size category. Protection shall extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet. See Sections 4.3.5, Design Procedures for Open Channels with Subcritical Flow, for further discussions on transitions.

Rock Material

Riprap failures generally result from undersized individual rocks in the maximum size range, improper gradation of the rock, and improper bedding for the riprap. Rock for riprap shall meet the requirements of the City's Standard Construction Specifications, Section 02190, Part 02.15.

Riprap for a stable channel lining shall be sized using Equation 4.8, which resulted from model studies by Smith and Murray (Smith 1965).

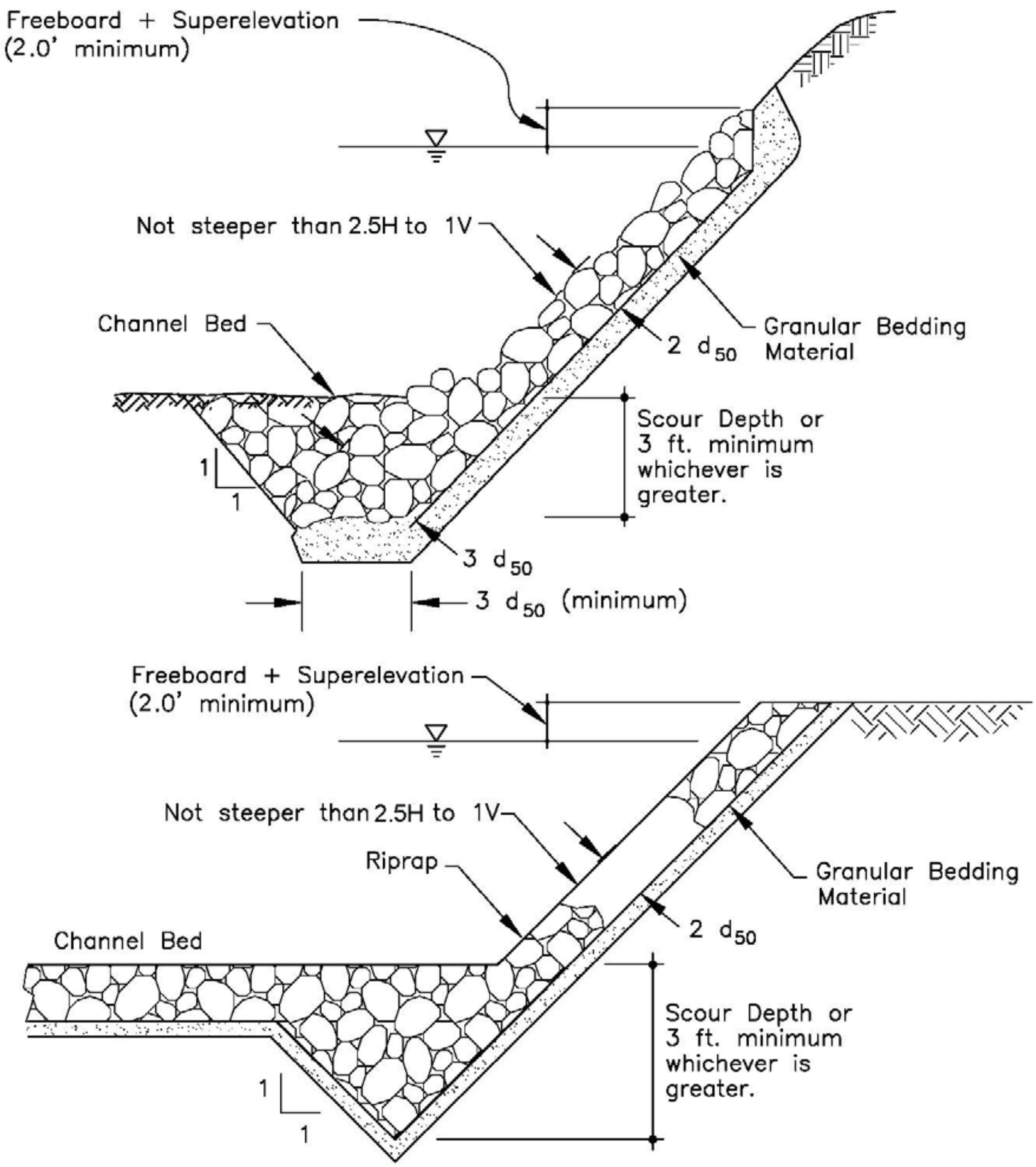
$$d_{50} = 0.05\, v^2 S^{0.34} / (S_s - 1)^{1.32} \qquad (4.8)$$

Where:

d_{50} = Rock size for which 50 percent of riprap by weight is smaller (ft)
v = Mean channel velocity (fps)
S = Longitudinal channel slope (ft/ft)
S_s = Specific gravity of rock (minimum S_s = 2.50) (dimensionless)

The riprap blanket thickness shall be at least 2.0 times d50 and shall extend up the side slopes to an elevation of the design water surface plus the calculated freeboard.

Figure 4-4
Typical Cross-Section for Riprap-Lined Channel



Bedding

Long term stability of riprap erosion protection is dependent upon proper bedding conditions. Many riprap failures are directly attributable to bedding failures. Gradations for granular riprap bedding are in the City's Standard Construction Specifications, Section 02190, Part 02.16.

Properly designed bedding provides a buffer of intermediate sized material between the channel bed and riprap to prevent movement of soil particles through the voids in the riprap. Three types of bedding are in common use: generic single-layer granular bedding, granular bedding based on the T-V (Terzaghi-Vicksburg) methodology, and drainage geotextile.

1. Generic single-layer granular bedding

The gradation of a single layer bedding specification is based on the assumption that the bedding will generally protect the underlying soil from displacement during a flood event. The single layer bedding design does not require any soil information, but in order to be effective covering a wide range of soil types and sizes, this method requires a greater thickness than the T-V method.

A single 12-inch layer of said granular bedding can be used except at drop structures. At drop structures, drainage geotextile must be added below the 12-inch layer of granular bedding.

2. Granular Bedding – T-V Design

The T-V design establishes an optimum granular bedding gradation for a specific channel soil. The specifications for the T-V reverse filter method relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

$$D_{15(\text{filter})} \leq 5d_{85(\text{base})} \tag{4.9}$$

$$4d_{15(\text{base})} \leq D_{15(\text{filter})} \leq 20d_{15(\text{base})} \tag{4.10}$$

$$D_{50(\text{filter})} < 25d_{50(\text{base})} \tag{4.11}$$

"D" refers to the filter grain size and "d" to the base grain size, and the subscripts refer to the percent by weight that is finer than the grain size denoted by either "D" or "d". For example, 15 percent of the filter material is finer than $D_{15(\text{filter})}$ and 85 percent of the base material is finer than $d_{85(\text{base})}$.

When the T-V method is used, the thickness of the resulting layer of granular bedding may be reduced to 6 inches. However, if a gradation analysis of the existing soils shows that a single layer of T-V Method designed granular bedding cannot bridge the gap between the riprap specification and the existing soils, then two or more layers of granular bedding shall be used. The design of the bedding layer closest to the existing soils shall be based on the existing soil gradation. The design of the upper bedding layer shall be based on the gradation of the lower bedding layer. The thickness of each of the two or more layers shall be 4 inches or more.

3. Drainage Geotextiles

Drainage geotextile (filter fabric) provides filtering action only perpendicular to the fabric. It has only a single equivalent pore opening between the channel bed and the riprap and a relatively smooth surface, which provides less resistance to stone movement. Therefore, filter fabric is not a complete substitute for granular bedding. Nonetheless, filter fabric can be an adequate replacement for granular bedding in many instances. Filter fabric provides adequate bedding for channel linings along uniform channels on mild slopes where leaching forces are primarily perpendicular to the fabric.

The use of filter fabrics in place of granular bedding should be restricted to slopes no steeper than 2.5H:1V, and such that the filter fabric replaces only the bottom layer in a multi-layer T-V Method granular bedding design. The granular bedding shall be placed on top of the filter fabric to act as a cushion when placing the riprap. Tears in the fabric greatly reduce its effectiveness, so direct dumping of riprap on filter fabric is not allowed during construction.

At drop structures and sloped channel drops, seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric. Seepage parallel with the fabric may be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric has to be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay can clog the openings in filter fabric preventing free drainage, which increases failure potential due to uplift. For this reason, a granular filter is often more appropriate bedding for fine silt and clay channel beds.

Grouted Riprap

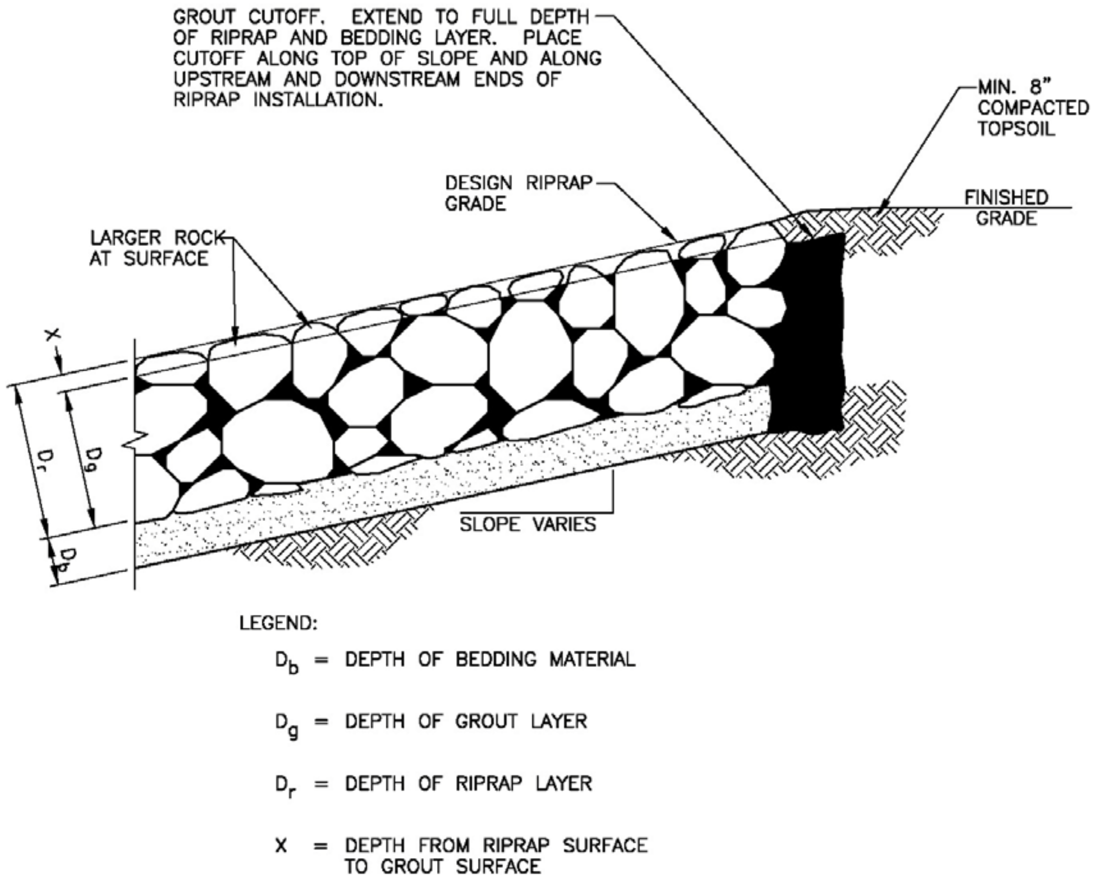
Grouted riprap provides a relatively impervious channel lining which is less subject to vandalism than loose riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low-flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning excess grout from the projecting rock with a wet broom prior to curing. A typical cross-section for grouted riprap lining is shown in Figure 4-5.

The rock used for grouted riprap differs from the standard gradation of riprap in that the smaller rock is removed to allow larger void spaces and greater penetration by the grout. Specifications for rock for grouted riprap are in the City's Standard Construction specification, Section 02190, Part 02.15. Riprap smaller than D_{50} = 12 inches shall not be grouted.

As the riprap layer is placed, a cutoff trench shall be excavated around the rock section at the top of the slope, and at the upstream and downstream edges. The trench shall be, at a minimum, the full depth of the riprap and bedding layer and at least 1 foot wide. This trench is filled with grout to prevent water from undermining the grouted rock mass.

After the riprap has been placed to the required thickness and the trench excavated, the rock is sprayed with clean water, which cleans the rock and allows better adherence by the grout. The rock is then grouted using a low pressure (less than 10 psi) grout pump with a 2-inch maximum diameter hose. Using a low pressure grout pump allows the work crew time to move the hose and vibrate the grout. Vibrating the grout with a pencil vibrator assures complete penetration and filling of the voids. After the grout has been placed and vibrated, a small hand broom or gloved hand is used to smooth the grout and remove any excess grout from the rock. The finished surface is sealed with a curing compound.

Figure 4-5
Typical Cross-Section for Grouted Riprap-Lined Channel



4.4.10 Wetlands-Vegetation-Bottom Channels

Wetland-bottom channels shall be designed in accordance with any special considerations for a particular site situation and the following:

Channel Geometry

1. The longitudinal channel slope shall be set so that the maximum permissible velocities are not exceeded. To prevent channel degradation, the channel slope shall be determined assuming there is no wetland vegetation on the bottom (i.e., "New Channel" condition). In addition to the velocity requirements, the Froude Number for the new channel condition shall be less than 0.7.
2. In selecting an appropriate channel bottom width, the design shall consider wetland mitigation requirements, constructability, channel stability and maintenance, multi-use activities, and low-flow channel width.

3. The maximum design depth of flow (outside of the low-flow channel area) should not exceed 5 feet for a 100-year flow of 1,500 cfs or less. For greater flows, excessive depths shall be avoided to minimize high velocities and for public safety considerations.
4. Side slopes shall not be steeper than 3(H):1(V).

Roughness Coefficients

The channel shall be designed for two flow roughness conditions. A Manning's roughness coefficient, assuming there is no growth in the channel bottom (New Channel condition), shall be used to set the channel slope. The "Mature Channel" condition occurs when wetland vegetation in the channel bottom has been established. The required channel depth including freeboard is determined assuming Mature Channel conditions.

A composite Manning's roughness coefficient shall be used for the New Channel condition design and the Mature Channel condition design. The composite Manning's roughness coefficient is determined by Equation 4.12 (Chow 1959):

$$n_c = (n_o^2 P_o + n_w^2 P_w)^{0.5} / (P_o + P_w) \qquad (4.12)$$

Where:

n_c = Manning's roughness coefficient for the composite channel
n_o = Manning's roughness coefficient for areas above the wetland area
n_w = Manning's roughness coefficient for the wetland area
P_o = Wetland perimeter of channel cross-section above the wetland area (ft)
P_w = Wetland perimeter of the wetland channel bottom (ft)

For grass-lined areas above the wetland area, use a Manning's roughness coefficient n_o = 0.035. Manning's roughness coefficients for the wetland area (n_w) can be obtained from Figure 4-6.

Low-Flow and Trickle Channel

Concrete trickle channels are not permitted in wetland bottom channels. Low-flow channels may be used when the 100-year flow exceeds 1,000 cfs. The design of the low-flow channel shall be according to Section 4.4.7, Grass-Lined Channels.

Channel Crossings

Whenever a wetland-bottom channel is crossed by a road, railroad or a trail requiring a culvert or a bridge, a grade-control structure shall be provided immediately downstream of the crossing to help reduce silting-in of the crossing. A 1-foot to 2-foot drop is recommended. The designer shall determine the hydraulics of the crossing and the drop structure, and design the structures to ensure the stability of the channel.

Life Expectancy

Wetland-bottom channels are expected to fill with sediment over time because the vegetation traps sediments carried by the flow. The life expectancy of such a channel depends primarily on the land use of the tributary watershed, and could range from 20 to 40 years before major channel dredging is needed. However, life expectancy can be dramatically reduced to as little as 2 to 5 years, if erosion in the tributary watershed is not controlled. Therefore, erosion practices need to be strictly controlled during new construction within the watershed, and all facilities need to be built to minimize soil erosion in the watershed to maintain a reasonable economic life of a wetland-bottom channel.

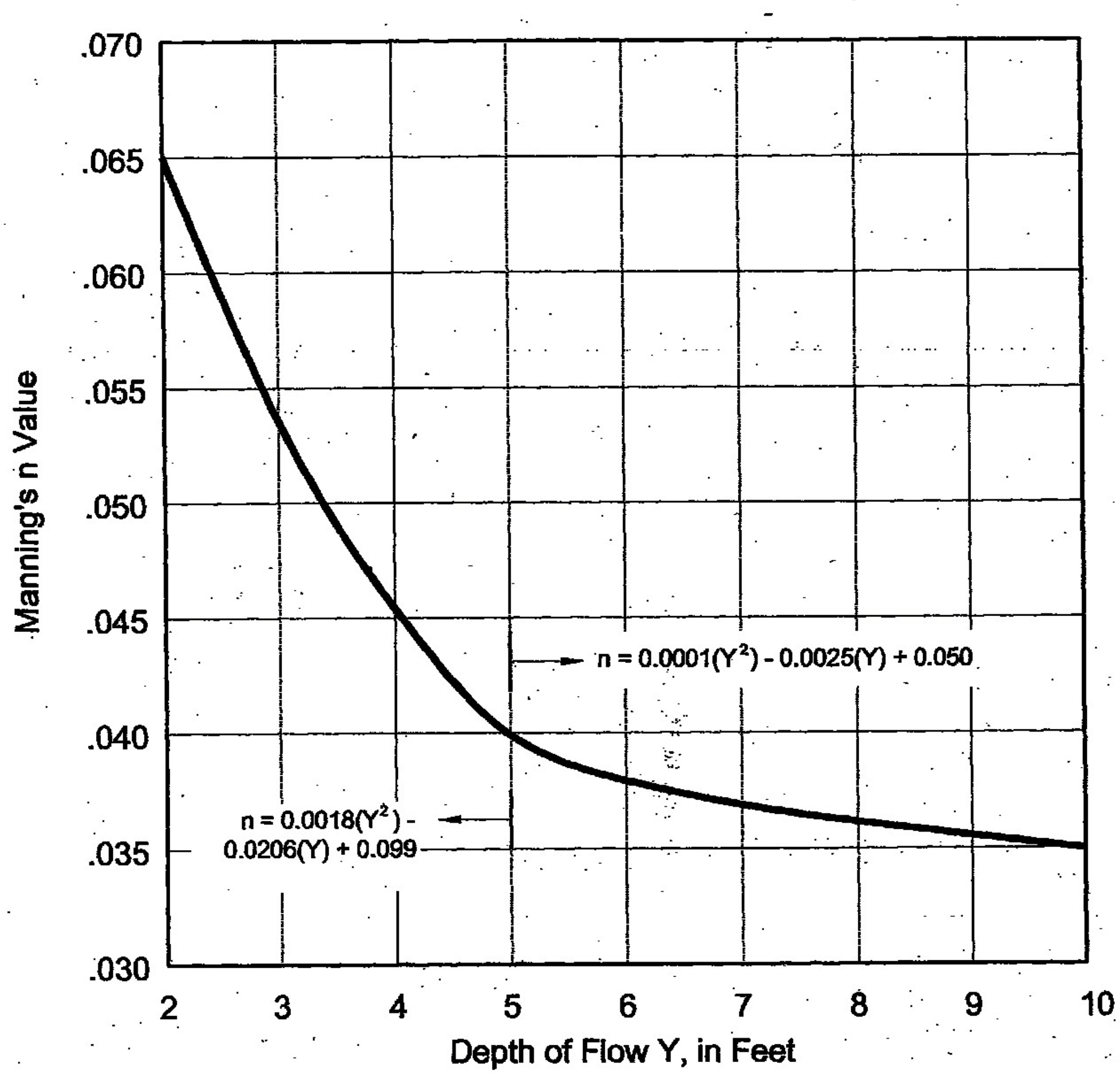
4.4.11 Other Lining Types

Other channel linings include all channel linings not discussed in the previous sections. These include composite-lined channels, which are channels in which two or more different lining materials are used (e.g., riprap bottom with concrete side slope lining), gabions, soil cement linings, synthetic fabric and geotextile linings, preformed block linings, reinforced soil linings, and flood walls (vertical walls constructed on both sides of an existing floodplain). The wide range of composite combinations and other lining types does not allow a discussion of all potential linings in this Drainage Criteria. For linings not discussed, documentation shall be submitted to support the use of the desired lining. The following items shall be addressed in the supporting documentation:

1. Structural integrity of the proposed lining.
2. Interfacing between different linings.
3. The maximum flow velocity under which the lining will remain stable.
4. Potential erosion and scour problems.
5. Access for operations and maintenance.
6. Long-term durability of the product under the extreme meteorological and soil conditions.
7. Ease of repair of damaged section.
8. Past case history (if available) of the lining system in other arid areas.
9. Potential groundwater mitigation issues (e.g., weepholes, underdrains).

These linings will be allowed if approved by the City. The City may reject the proposed lining system due to operation, maintenance, and public safety considerations.

Figure 4-6
Manning's n for Composite Channel and Low-Flow Section Design



4.4.12 Minor Artificial Channels and Ditches

A minor channel is defined as a drainageway with a contributing drainage area of less than 160 acres. Additional flexibility and less stringent standards may be allowed for minor channels. Only the differences in a channel type's design as a minor channel versus that of a major channel are presented in this section.

For grass, concrete and riprap-lined swales and channels with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirement is 6 inches.

For grass, wetland-bottom, concrete, and riprap-lined swales and channels, the minimum radius with a 100-year runoff of 20 cfs or less shall be 25 feet.

Roadside Ditches

Roadside ditches shall be designed to convey the Minor design storm. Where design velocities will create unstable conditions, erosion protection or grade-control structures shall be provided. Major storm event flows shall be contained within the public right-of-way or easements. Roadside ditches shall also be designed within the following limitations:

1. Minimum depth – 24 inches
2. Maximum side slopes – 3(H):1(V)
3. Maximum bottom width – 4 feet

4.5 CHANNEL APPURTENANCES

All improved channels shall be designed to include the following appurtenances.

4.5.1 Maintenance Access Road

A maintenance access road with an all-weather surface and a minimum passage width of 12 feet shall be provided along the entire length of all improved channels with 100-year design capacity equal to or greater than 100 cfs. The maintenance access road shall be located above the Minor event water surface elevation, and shall connect to public streets. Maintenance access road profiles need to be shown for all critical facilities such as roadway and stream crossings.

4.5.2 Safety Requirements

The following safety measures are required for concrete-lined channels. Similar safety measures may be required for all other channels.

1. A six-foot high galvanized-coated chain link or comparable fence shall be installed to prevent unauthorized access. The fence shall be located at the edge of the ROW or on the top of the channel lining. Gates, with top latch, shall be placed at maintenance access points or 1,320-foot intervals, whichever is less.
2. Ladder-type steps shall be installed at intervals of not more than 1,200 feet, and shall be staggered on alternating sides of the channel to provide a ladder every 600 feet. The bottom rung shall be placed approximately 12 inches vertically above the channel invert.

4.5.3 Low-Flow Grade Control Structures

Urbanization causes more frequent and sustained flows, and trickle/low-flow channels become more susceptible to erosion even though the overall floodplain may remain stable and able to resist major flood events. Erosion of the low-flow channel, if left uncontrolled, can cause degradation and destabilization of the entire floodplain and water quality problems. Low-flow check structures provide control points and establish stable bed slopes within the base-flow channel. Check structures can be small versions of the drop structures described in Section 9, Hydraulic Structures, or in many instances simply control sills across the floodplain. Low-flow check structures are not appropriate in completely incised floodplains or very steep channels.

In addition, low-flow structures inherently permit a considerable amount of sediment to be transported downstream as urbanization takes place, which negatively impacts stormwater quality. Therefore, complete channel stabilization using drop structures in conjunction with channel bed and bank stabilization measures may be required at the time of development.

Control Sill Grade-Control Structures

Another type of check structure that can be used to stabilize low-flow channels within wide, relatively stable floodplains is the control sill shown in Figure 4-7. The sill can be constructed by filling an excavated trench with concrete, if soil conditions are acceptable for trenching, or forming a simple wall if a trench will not work.

The sill crosses the low-flow channel and shall extend a significant distance into the adjacent floodplain on both sides. The top of the sill conforms to the top of the ground at all points along its length. Riprap or other erosion-control methods can then be added as erosion occurs. The basic design steps are:

1. Determine a stable slope as described above.
2. Determine spacing of the sills based on the difference in slope between the natural and projected stable slope and the amount of future drop to be allowed (not to exceed 3 feet).

4.6 DRAINAGE EASEMENTS

4.6.1 Maintenance and Access Easements

To provide maintenance access to drainage facilities to be maintained by the City outside of already established public right-of-way, an easement or tract equal to the channel top width, including freeboard, plus 20 feet shall be conveyed to the City. All channels to be maintained by the City shall be provided with maintenance access from adjacent public right-of-way through tracts or easements as required herein.

Maintenance access may be aligned along the top of channels or within the channel above the level of Minor storm flows.

4.6.2 Drainage Easements on Residential Lots

Drainage across the rear or sides of residential lots shall be provided as follows.

Where the average lot size is less than 15,000 square feet, dedicated rear/side lot drainage easements shall not be required where 2 lots or less drain across a downstream lot. Where more than 2 lots drain across a downstream lot, the drainage area shall be dedicated to the City of Gillette as an easement or other approved dedication.

Where the average lot size is 15,000 square feet or greater and where flows are concentrated or as otherwise deemed necessary, drainage areas shall be dedicated to the City of Gillette as an easement or other approved dedication.

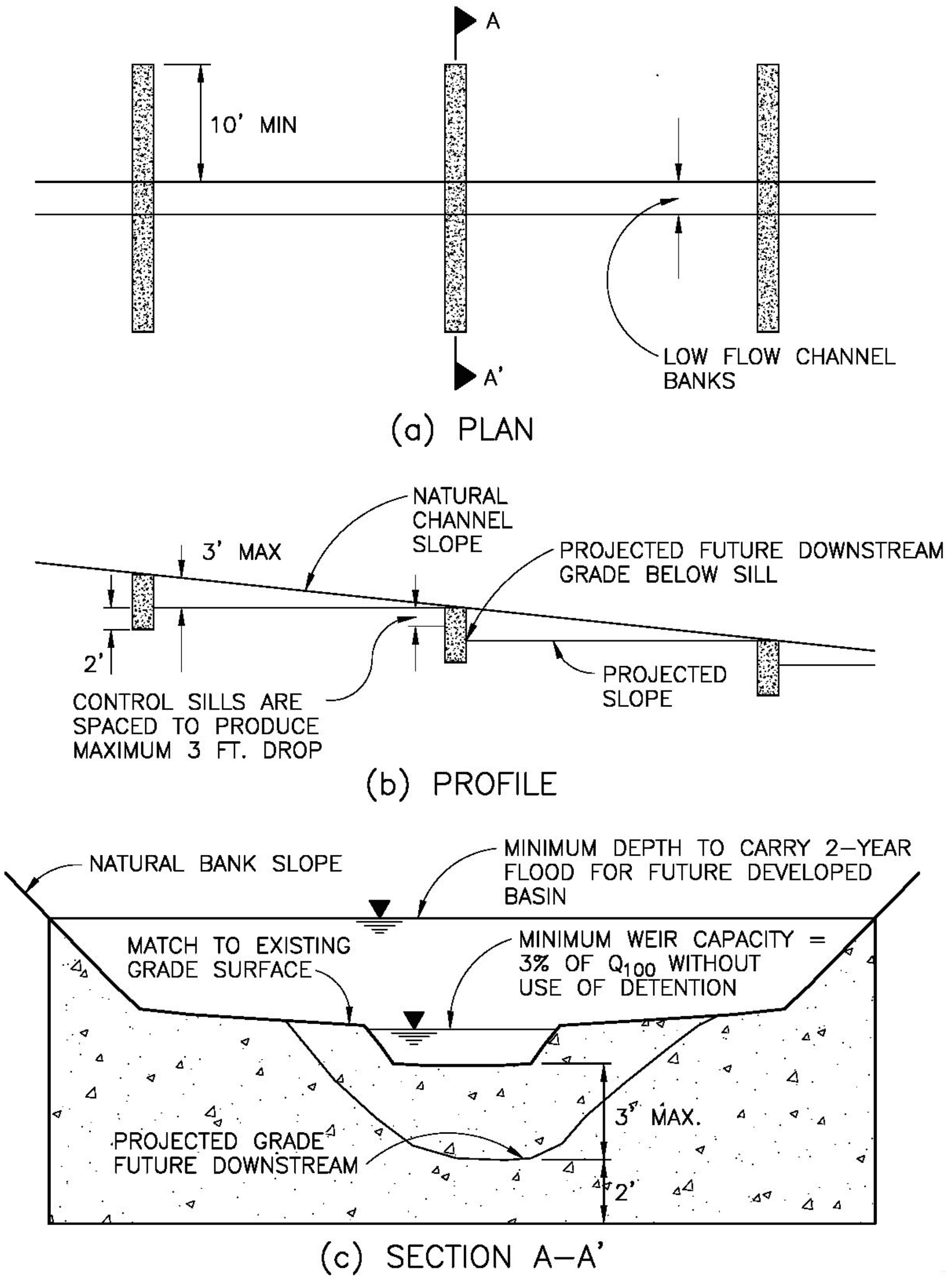
The conveyance system through a residential drainage easement shall be in accordance with this Section.

4.7 CHECKLIST

All of the design criteria in this section must be followed. Several key considerations that the designer must address include:

1. Check flow velocity with low retardance "n" factor and capacity with high retardance "n" factor.
2. Check Froude Number and critical flow conditions.
3. Grass channel side slopes must be 4:1 or flatter.
4. Show EGL and water surface profile on design drawings.
5. Consider all backwater conditions (i.e., at culverts) when determining channel capacity.
6. Check velocity for conditions without backwater effects.
7. Provide adequate freeboard.
8. Provide adequate ROW for the channel and continuous maintenance access.

Figure 4-7
Control Sill Grade-Control Structure



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SECTION FIVE
STORM DRAINS

5.1 INTRODUCTION

This section presents criteria for design of storm drain systems.

Storm drains are used to convey runoff in locations where streets or other drainage facilities exceed their designated capacity or are otherwise unable to drain. The most common means for collecting water into a storm drain is the street inlet (see Section 6, Inlets). However, water may also enter the system through grated area inlets, culvert-type inlets (typically for the conveyance of drainage channel flow into the storm drain), pump stations, or other entry points. The design of a storm drain system is dependent on topography, street rights-of-way and drainage easements, collection and conveyance of runoff from multiple locations, existing and proposed structures and utilities, outfall locations, and local hydrology.

Occasionally, inlets and storm drains must be sized to convey the entire Major storm event flow. Two examples of this situation are:

1. Locations where street flow is not in the desired direction and there is no other feasible drainage solution (such as closed basins – natural ponding areas).
2. Locations where the standard allowable major storm street capacities do not apply, such as negative slopes outside the curb but within the right-of-way.

Methods for determining peak runoff rates are in Sections 2, Rainfall, and 3, Runoff Analysis, of these Drainage Criteria.

5.1.1 Allowable Capacity and Velocity

Generally, storm drain improvements shall be designed so that Minor storm peak flows do not exceed curb-full street capacities and Major storm flows do not exceed allowable flow depths in streets (see Section 7, Streets). To maintain flow depths less than those allowed during the Major storm event, it may be necessary to convey flows greater than the Minor storm. Storm drains shall also be located to drain low points.

Storm drains shall be designed to convey their flow-full capacity during the Minor storm event within the storm drain without surcharging. Storm drains may be designed to surcharge during Major storm events; however, surcharging shall not cause the HGL elevation to be higher than 1 foot below the adjacent gutter flow line elevation. The HGL at inlets, including lateral losses, shall be at least 1 foot below the gutter flow line elevation adjacent to the inlet. Mean pipe velocities shall be between 2 fps and 15 fps for Minor storm events. Mean velocities may be as high as 20 fps during the Major storm event with special consideration given to the integrity of pipes and manholes.

The minimum diameter for storm drain trunk lines shall be 18 inches. The minimum diameter for lateral pipes connected to storm inlets shall be 15 inches. Laterals are defined as a storm drain line that has a maximum of two inlets connected to it. For non-circular pipe sections, the minimum dimension shall be 12 inches. Pipe sizes shall increase in a downstream direction to avoid trapping debris and maintain design capacities.

5.1.2 Vertical and Horizontal Alignment

Storm drain systems shall be placed in public ROW or easements as follows:

Vertical Alignment

Vertical bends are not permitted. When pipe sizes increase in a downstream direction, pipe crowns shall be set at the same elevation.

Minimum and maximum cover are determined by the size, material, and class of pipe, as well as the characteristics of the cover material and the expected surface loading. The designer shall consult the appropriate data sources to include:

- Wyoming Department of Transportation Standard Plans 603-1A, Pipe Fill Height Chart and Installation Details
- Concrete Pipe Design Manual (ACPA)
- Handbook of Steel Drainage and Highway Construction Products (AISI)
- Pipe Manufacturer Specifications

Storm drains crossing under railroads and highways must comply with any cover requirements specified for culverts (Section 8, Culverts) and requirements of the railroad and Wyoming Department of Transportation.

Pipes installed under any driving or parking area shall be designed for HS-20 minimum live load. It is recommended that storm drain trunk mains (any storm drain to which laterals connect) have a minimum cover of 36 inches over the top of the pipe, including any pavement thickness, or be in accordance with manufacturer's specifications for load cases.

Horizontal Alignment

Storm drain mains are to be located under the curb or as approved by the City Engineer. Curved alignments shall only be permitted on a case-by-case basis by the City Engineer.

Utility Clearances

The designer shall consult with the most recent versions of the following documents to ensure compliance with the most restrictive (largest) utility clearance values applicable to the subject location:

- City of Gillette Design Standards and Construction Specifications for Underground Utilities – Waterlines, Sanitary Sewers, Storm Drains.
- Utility clearance requirements set forth by the City.

Pipe encasement may be required in some locations where minimum utility clearances cannot be met. Standards for the design and installation of a bore casing pipe or cement treated fill (pipe saddle) can be found in the Standard Drawings of the City of Gillette Design Standards.

5.2 HYDRAULIC DESIGN

This section presents the hydraulic methods used to calculate storm drain capacities and thereby to design a storm drain system. The majority of the methods in this section are adapted from those presented in *HEC-22 (Urban Drainage Design Manual)* and *HDS-4 (Introduction to Highway Hydraulics).*

5.2.1 Gravity Flow Analysis

Initial storm drain design is completed by selecting pipe sizes based on "just full" capacity. This means that the drain capacity is calculated using open-channel (non-pressurized) flow computations. Starting at the uppermost reach of the storm drain (at the first inlet), apply Manning's equation for each segment of the storm drain. A segment is a reach of pipe with a junction, transition, grade change, horizontal bend, or pipe size change at each end.

$$Q_f = 1.49\, A_f R_f^{2/3} S_o^{1/2} / n \qquad (5.1)$$

Where:

Q_f = Full flow discharge (cfs)
n = Manning's roughness coefficient (Table 5-1)
A_f = Full flow area = $\pi D^2/4$ for circular pipes (ft^2)
R_f = Full flow hydraulic radius = D/4 for circular pipes (ft)
S_o = Pipe slope ($S_o = S_f$ for full flow) (ft/ft)
D = Pipe diameter (ft)

Equation 5.2 is a form of Manning's that can be used to directly solve for the minimum required pipe diameter for circular pipes. For noncircular pipes, this equation provides the equivalent diameter based on flow area. Initial pipe size, D_i (ft), is based on the peak design flow for that pipe segment, Q_P (cfs).

$$D_i = (2.16 n Q_P / S_o^{1/2})^{3/8} \qquad (5.2)$$

A range of Manning's *n* values for many pipe materials and configurations as developed by Chow in 1959 and Normann in 1985 (adapted from tables found in HDS-4 and HEC-22) is provided in Table 5-1. For purposes of storm drain design, hydraulic roughness shall be specified by the largest Manning's *n* value in the provided range. The designer may choose to use a higher Manning's *n* value if conditions warrant.

Table 5-1
Roughness Coefficients (Manning's n) for Storm Drain Conduits

Type of Conduit	Interior Wall Description	Manning's n
Concrete Pipes	Smooth	0.011-0.013
Concrete Boxes		
Wood Forms	Smooth	0.012-0.014
Steel Forms	Smooth	0.012-0.013
Spiral-Rib Metal Pipes	Smooth	0.012-0.013

Table 5-1
Roughness Coefficients (Manning's n) for Storm Drain Conduits

Type of Conduit	Interior Wall Description	Manning's n
High-Density Polyethylene (HDPE)	Smooth	0.009-0.015
	Corrugated	0.018-0.025
Polyvinyl Chloride (PVC)	Smooth	0.009-0.012
Cast-Iron Pipe, uncoated		0.013
Steel Pipe		0.009-0.013

Source: Adapted from HDS-4 & HEC-22.

Note: The Manning's n values in this table were obtained in the laboratory and are supported by the provided sources. Actual field values for older existing conduits may vary depending on the effects of abrasion, corrosion, deflection and joint conditions (HDS-4). The designer should take these potential issues into account when selecting roughness values.

5.2.2 Pressure Flow Analysis

Following the initial "just-full" storm drain design, the system is analyzed using energy-momentum theory to account for specific energy losses. This method allows for the calculation of HGL and EGL for a given storm drain line by starting with the water surface elevation of the outfall and working upstream, accounting for all losses due to pipe friction, manholes, transitions, bends, junctions, and pipe entrances and exits. In cases where pressure flows exists, certain limitations exist on the maximum elevation of the EGL in relation to the ground surface (finished grade). Compliance with minimum and maximum flow velocities is based on peak design flow in the final selected pipe size for each segment.

Energy-momentum theory is based upon the concept that energy, typically expressed in hydraulics as "head" in a linear dimension such as feet, is conserved along a given conduit segment. For a segment where *A* is the upstream end and *B* is downstream, the steady-flow energy equation can be expressed as:

$$z_A + p_A/\gamma + v_A^2/2g + h_p = z_B + p_B/\gamma + v_B^2/2g + \Sigma h_L \qquad (5.3)$$

Where:

z = Invert elevation above any horizontal datum (ft)
p = Fluid pressure (lbf/ft^2)
γ = Specific Weight of Water $\cong$ 62.4 lbf/ft^3
v = Flow velocity (fps)
h_p = Head added by a pump (if applicable) (ft)
Σh_L = Sum of head losses in Segment A - B as calculated per the methods prescribed in this section

Each term in the energy equation, and thus the sum of the formula, has a linear dimension (ft). Each term represents the hydraulic head contributed to the total energy head by that term. For instance, the third term, $v^2/2g$ is the velocity head. The EGL elevation at a given point is equal to:

$$EGL = z + p/\gamma + v^2/2g \qquad (5.4)$$

and the HGL elevation is simply the EGL minus the velocity head:

$$HGL = EGL - v^2/2g \text{ and conversely } EGL = HGL + v^2/2g \quad (5.5)$$

In cases where outfall water surface is equal to or higher than the outlet flow elevation, the EGL and HGL are assumed to be equal, i.e., velocity is zero at the downstream point where calculations start. However, if the outfall water surface is lower than the outlet pipe flow elevation, the latter value is used as the outlet HGL. The outfall water surface elevation used must be determined coincident with the time of peak flow from the storm drain.

The HGL at the next structure (e.g., manhole) is determined by the equations presented in Table 5-2. The equations are separated by HGL at the pipe inlet downstream of the manhole and the pipe outlet at the inlet to the manhole. For non-surcharged flow (less than 80% pipe depth), the free water surface at the pipe inlet (downstream end of the manhole) is added to head loss across the manhole to find the pipe outlet HGL (upstream end of the manhole).

Table 5-2
Equations for Determining HGL

Surcharge Conditions	Outlet Submergence	HGL in Manhole/Junction	At	Equation Number
$d_n/D > 0.80$	N/A	$= HGL_{Pipe\ Outlet} + h_f + h_{minor}$	Pipe Inlet (D/S from MH)	5.6
$d_n/D > 0.80$	N/A	$= HGL_{Pipe\ Inlet} + h_{mh}$	Pipe Outlet (U/S from MH)	5.7
$d_n/D \le 0.80$	Unsubmerged	$= WSE_{Pipe\ Inlet}$	Pipe Inlet (D/S from MH)	5.8
$d_n/D \le 0.80$	Unsubmerged	$= WSE_{Pipe\ Inlet} + h_{mh}$	Pipe Outlet (U/S from MH)	5.9
$d_n/D \le 0.80$	Submerged	= Larger of Equations 5.6 and 5.7 OR = Larger of Equations 5.8 and 5.9		

Where:

d_n = Normal flow depth in pipe (ft)
$HGL_{Pipe\ Oulet}$ = Larger of tailwater elevation, flow depth elevation at pipe outlet, and HGL at next downstream pipe inlet
$WSE_{Pipe\ Inlet}$ = Free water surface elevation at pipe inlet
h_f, h_{mh}, h_{minor} = Head losses as described in this section

Occasionally, design flow through a pipe may be not only gravity-flow (nonsurcharged) but also supercritical. Pipe losses (h_f and h_{minor}) in a supercritical pipe section are not carried upstream (HEC-22). In locations where two adjoining pipe segments flow in supercritical conditions, manhole losses are also ignored for that line. The designer shall be careful to include these losses where only one of the pipes on the segment under investigation contains supercritical flow.

Inlet pipes to a manhole must occasionally have an invert significantly above that of the outlet pipe. In locations where the outlet pipe water surface elevation (or HGL if pressure flow) is below the invert of an inlet pipe, that inlet pipe is treated as an outfall pipe. In this case, the outfall water surface elevation is always lower than the pipe outlet water level, so the latter elevation is used for the initial HGL of the new upstream reach. The outflow pipe from the manhole in such a situation acts as a culvert under either inlet or outlet control. See Section 8, Culverts, and/or Federal Highway Administration (FHWA) *Hydraulic Design of Highway Culverts* (HDS-5) for information regarding the computation of an HGL at the manhole and calculation of head loss due to a culvert inlet.

5.2.3 Computer Hydraulic Modeling

Since the storm drain system design process is iterative, computer programs are now commonly used to design and/or model proposed and existing storm drainage networks. HGL and EGL calculations may be prepared using computer software subject to review and approval by the City. It is recommended that the designer consult with the City before using any software that is newly released or has not already been broadly accepted by the engineering community.

5.2.4 Pipe Friction Losses

Pipe friction is a significant source of energy dissipation in storm drains, whether in gravity-flow or pressure-flow conditions. For the former, friction slope (S_f) can be assumed to be equal to the slope of the pipe invert (S_o). For pipes with a surcharge flow condition ($d_n/D > 0.80$), the equations below define friction slope (units for variables are the same as in Equation 5.1 when using English units).

$$S_f = n^2 V_{avg}^2 / K_Q R^{4/3} \tag{5.6}$$

Where:

K_Q = 2.21 for English Units; 1.0 for SI Units

$$S_f = (nQ_{avg} / K_Q D^{8/3})^2 \tag{5.7}$$

Where:

K_Q = 0.46 for English Units; 0.312 for SI Units

Since flow rate and cross-sectional area typically remain constant through one segment of pipe, average velocity can be assumed to equal flow rate divided by flow area. Where flow rate and/or pipe size changes within one segment (such as at a pipe transition without a manhole or a no-access junction), this velocity is the average of those calculated at the ends of the pipe segment (Linsley 1992).

Once the friction slope is known, pipe friction head loss is calculated by multiplying the friction slope by the pipe segment length:

$$h_f = S_f L \tag{5.8}$$

5.2.5 Preliminary Head Losses

To better account for energy losses that will occur in the system, the designer may choose to calculate preliminary head losses through inlet and manhole junctions. Application of these approximate losses allows for better estimation of required pipe sizes during the initial design process, expediting the preliminary and final design phases. *HEC-22* presents Equation 5.9 and Table 5-3 for calculating approximate junction losses:

$$H_{ah} = K_{ah} \left(v_o^2/2g\right) \qquad (5.9)$$

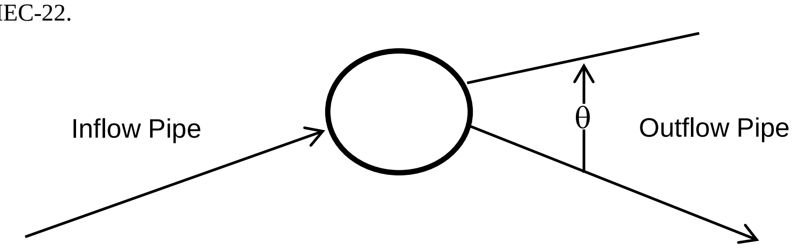
Where:

H_{ah} = Preliminary junction head loss estimate (ft)
K_{ah} = Head loss coefficient (Table 5-3)
v_o = Flow velocity = Q_P / A_f (fps)
g = Gravitational acceleration (32.2 ft/sec^2)

Table 5-3
Preliminary Head Loss Coefficients (for Conceptual/Initial Design ONLY)

Structure Configuration	Coefficient, K_{ah}
Inlet – Straight Run	0.50
Inlet – Angled Through (Θ)	
90°	1.50
60°	1.25
45°	1.10
22.5°	0.70
Manhole – Straight Run	0.15
Manhole – Angled Through (Θ)	
90°	1.00
60°	0.85
45°	0.75
22.5°	0.45

Source: HEC-22.



5.2.6 Manhole Junction Losses

This section describes the Energy-Loss Method (FHWA) as presented in HDS-4 for calculation of approximate head loss through a manhole. This method applies to any junction of two or more pipes accessible by a manhole.

For each manhole, the designer must first calculate the adjusted head loss coefficient (K) and head loss in the manhole (h_{mh}) by using Equations 5.10 and 5.11:

$$K = K_o C_D C_d C_Q C_p C_B \tag{5.10}$$

$$h_{mh} = K(v_o^2/2g) \tag{5.11}$$

The initial head loss coefficient and coefficient correction factors are calculated using the equations presented below. Note that some correction factors do not apply to all manhole configurations. These non-applicable factors are set to unity.

Initial Head Loss Coefficient (K_o)

The initial head loss coefficient is determined by Equation 5.12:

$$K_o = 0.1b\,(1-\sin\theta) / D_o + 1.4\,(b/D_o)^{0.15} \tag{5.12}$$

Where:

θ = Angle between inflow and outflow pipes (≤ 180°)
b = Manhole or junction diameter at water level (ft)
D_o = Outlet pipe diameter (ft)

Correction Factor for Pipe Diameter (C_D)

Equation 5.13 applies to pressure flow when the ratio of water depth in the manhole above the outlet pipe invert to outlet pipe diameter is greater than 3.2 ($d_{mho} / D_o > 3.2$).

$$C_D = (D_o/D_i)^3 \tag{5.13}$$

Where: D_o and D_i = Outlet and inlet pipe diameter (ft), respectively.

Correction Factor for Flow Depth (C_d)

Equation 5.14 applies to gravity and low-pressure flow when the ratio of water depth in the manhole above the outlet pipe invert to outlet pipe diameter is less than 3.2 ($d_{mho} /D_o < 3.2$).

$$C_d = 0.5(d_{mho}/D_o)^{0.6} \tag{5.14}$$

For purposes of this calculation, d_{mho} is approximated as the vertical distance from the outlet pipe invert to the HGL at the upstream end of the outlet pipe.

Correction Factor for Relative Flow (C_Q)

Equation 5.15 applies to manholes with three or more pipes entering the structure at similar elevations (one of these pipes will be the outlet pipe). This correction factor does not apply to the effects of inflow pipes with flowlines far enough above the outlet pipe to qualify as plunging flow (see Correction Factor for Plunging Flow).

$$C_Q = (1-2\sin\theta)(1- Q_i/Q_o)^{0.75} + 1 \tag{5.15}$$

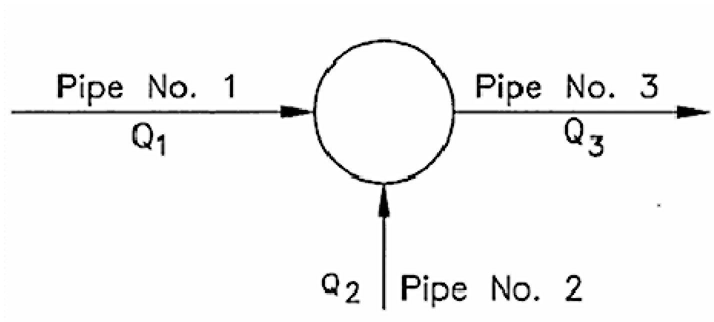
Where:

Θ = Angle between inflow pipe of interest and the outflow pipe

Q_i = Flow in the inflow pipe of interest

Q_o = Flow in the outflow pipe

The "pipe of interest" is the inlet pipe to the manhole on the line being investigated. This factor accounts for streamline interference by flow from other pipes entering the manhole. See figure below for an illustration of the relative flow effect.



Correction Factor for Plunging Flow (C_p)

Equation 5.16 applies to manholes with an inflow pipe of interest that is affected by plunging flow from another inflow pipe with a higher flowline. The factor does not apply to the line with the pipe that is discharging the plunging flow, and applies only when the height of the plunging-flow pipe flowline above the outlet pipe center (h) exceeds the manhole water depth above the outlet pipe invert ($h > d_{mho}$).

$$C_p = 1 + 0.2h\,(h-D_o) / D_o^2 \qquad (5.16)$$

Where:

h = Vertical distance of plunging flow (ft)

d_{mho} = Water depth in manhole above outlet pipe invert (ft)

D_o = Outlet pipe diameter (ft)

A common application of this correction factor occurs at locations where inlets convey intercepted flow directly (vertically) to the storm drain main line (drop inlets) or where laterals enter a manhole well above the main line invert.

Correction Factor for Benching (C_B)

This applies to all flow conditions. See Figure 5-1 and Table 5-4 for proper correction factors. As indicated in Table 5-4, benching in manholes significantly reduces head loss due to outlet inefficiency, especially in unsubmerged conditions. Note that in this case, the submerged pressure-flow factors do not apply until flow depth in the manhole has exceeded 3.2 times the outlet pipe diameter. Therefore, for depths between free surface (gravity) flow and full pressure-flow conditions ($1.0 > d_{mho}/D_o < 3.2$), the designer shall use a linear interpolation to compute the benching correction factor.

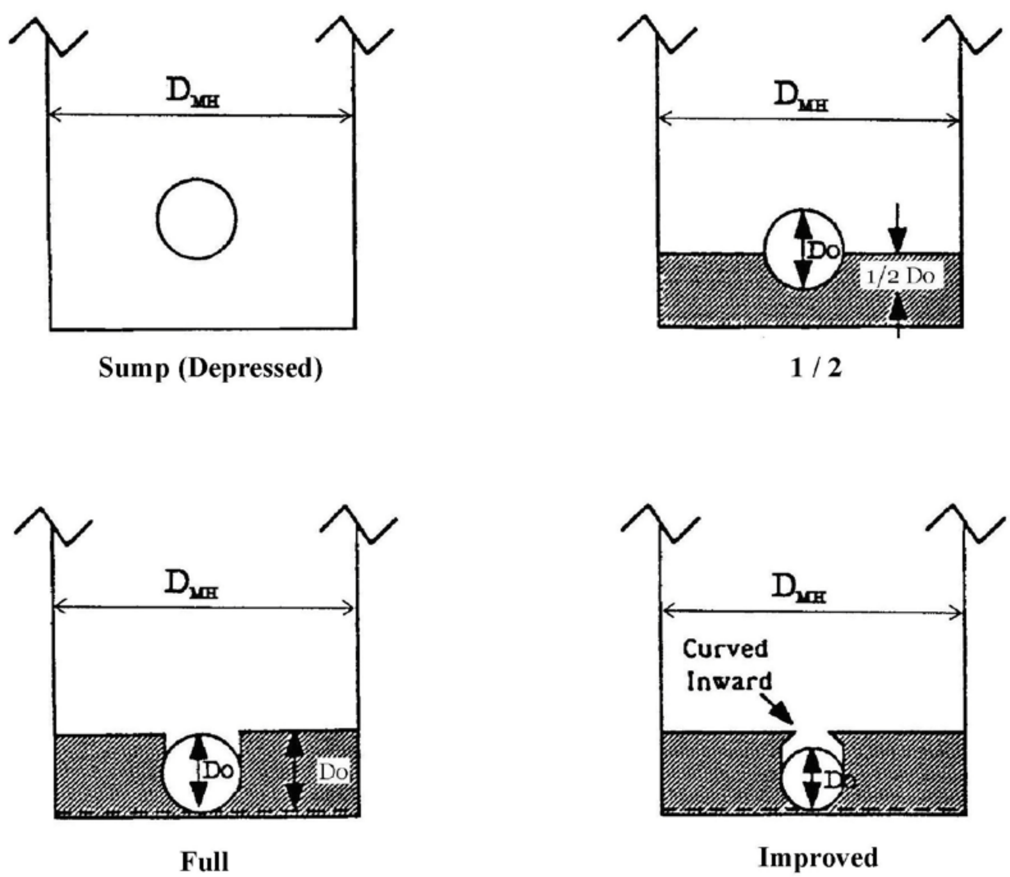
Table 5-4
Benching Correction Factors

Bench Type (See Figure)	Outlet Pipe Conditions	
	Fully Submerged, Pressure Flow[1]	Unsubmerged, Free Surface Flow[2]
Depressed	1.00	1.00
Benched: ½ Pipe Diameter	0.95	0.15
Benched: 1 Pipe Diameter	0.75	0.07
Improved Bench	0.40	0.02

[1]Applies for $d_{mho}/D_o \geq 3.2$

[2]Applies for $d_{mho}/D_o \leq 1.0$

Figure 5-1
Types of Manhole Benching



5.2.7 Minor Pipe Losses

This section describes methods used for the calculation of head losses caused by pipe transitions (expansions or contractions), no-access junctions, on-grade inlets, and exits (outlets). For new system design, most minor pipe losses will occur in a manhole and should be calculated using the methods outlined above.

Minor losses are added together for a given pipe segment using Equation 5.21:

$$h_{minor} = \Sigma\, h_{L,minor} = h_e + h_c + h_j + h_i + h_o \qquad (5.17)$$

Transition Losses (h_e and h_c)

Transition losses occur when pipe size is changed at a location other than a manhole. Expansions may be necessary due to changes in flow rate or slope, and are only allowed in a manhole. Contractions are locations where pipe size is decreased, and are allowed only through a variance. For evaluating existing conditions where there are no-access contraction or expansions, the user is referred to HEC-22.

No-Access Junctions (h_j)

This term applies to head loss associated with locations where a lateral pipe connects to a larger trunk pipe without the use of a manhole structure. These junctions are not recommended for trunk lines of less than 48 inches in diameter, but it is sometimes physically or economically inefficient to place a manhole at every junction location. At locations where more than one lateral joins the trunk line a manhole is required. The head loss at no-access junctions is related to the relative flows and velocities of all three pipes, the angle between the lateral and trunk pipes, and the cross-sectional area of the trunk pipe.

$$h_j = (Q_o v_o - Q_i v_i - Q_L v_L \cos\theta) / [0.5g\,(A_o + A_i)] + h_{vi} + h_{vo} \qquad (5.18)$$

Where:

Q_o, Q_i, Q_L = Outlet, Inlet, and Lateral flow rates
v_o, v_i, v_L = Outlet, Inlet, and Lateral velocities
h_{vo}, h_{vi} = Outlet and Inlet velocity heads ($v^2/2g$)
A_o, A_i = Outlet and Inlet cross-sectional areas
θ = Angle of lateral with respect to outflow pipe

On-grade/Culvert Inlets (h_i)

In some locations, water may enter a storm drain system from a drainage channel, overflowing pond, or other conveyance with a flowline approximately equal to that of the storm drain inlet. These are hydraulically equivalent to culvert inlets. The coefficient K_i in Equation 5.25 is equal to the culvert entrance loss coefficient K_e provided in Section 8, Culverts, Table 8-2, Entrance Loss Coefficient, of these Drainage Criteria. (Note that K_e represents the expansion loss coefficient in Section 5, Storm Drains.)

$$h_i = K_i\,(v^2/2g) \qquad (5.19)$$

Where: K_i = On-grade inlet coefficient (Table 8-2)

Outlets/Pipe Exits (h_o)

This term applies to pipe outfalls into an open waterway, such as an open channel, detention/retention basin, or other receiving waters. Outlets that discharge into a body of water with essentially zero velocity in the direction of the storm drain exit lose all velocity (one velocity head). This includes outlets perpendicular to an open channel and all submerged outlets. The storm drain flow is also assumed to lose all velocity when it discharges to open air and plunges to the receiving waters.

$$h_o = v_o^2/2g - (v_d^2/2g) \tag{5.20}$$

Where:

v_o = Flow velocity at storm drain outfall
v_d = Flow velocity in receiving waters (in the direction of storm drain flow)

5.2.8 Storm Drain Outlets

Proper design of storm drain outlets is necessary to minimize erosion at the outfall location and to protect public safety. Adequate erosion protection shall be provided at all storm drain outlets in the form of riprap or concrete stilling basins, designed in accordance with Section 9, Hydraulic Structures, of these Drainage Criteria.

Flared end sections shall be provided for storm drain outfalls 48 inches or less in diameter. Headwalls and wingwalls associated with storm drain outlets shall be provided with guardrails, handrails, or fencing in conformance with City building codes and roadway design safety requirements. Handrails or fencing shall be required in all areas where the drop from the headwall or wingwall exceeds 30 inches. The height of the handrail shall be 42 inches for pedestrian walkways or open areas and 54 inches for bicycle traffic (AASHTO 2002).

5.2.9 Manholes

Manholes or junction boxes are necessary to provide maintenance and inspection access to the storm drain. When designed correctly, they also provide more hydraulically efficient pipe junctions and other transitions. All manhole lids must bear the word “storm” for identification purpose.

Manholes shall be placed at all changes in main-line pipe size or grade, junctions where a lateral joins the main-line alignment, main-line vertical drops (drop manhole), and mainline direction changes.

Pipes of 48 inches or larger diameter do not necessarily require manholes at all locations specified above. However, manholes in addition to those required by standard maximum spacing may be stipulated by the City Engineer.

Tables 5-5 and 5-6 give the maximum spacing and minimum sizes for manholes. Larger manhole diameters or junction boxes may be required when storm drain alignments are not straight through or when lateral pipes also enter manholes. Noncircular pipes shall be converted to equivalent diameters based on pipe area.

Table 5-5
Maximum Allowable Manhole Spacing

Storm Drain Diameter	Maximum Distance
18” to 36”	400’
42” to 60”	500’
66” and Larger	600’

Table 5-6
Minimum Allowable Manhole Sizes

Storm Drain Diameter	Manhole Diameter
18” to 24”	4’
27” to 36”	5’
42”	6’
48” and larger	Junction box or Tee

Inlet or junction box is acceptable in lieu of a manhole where access is provided. The EGL for the all design flows must be at or below the manhole rim.

Manhole depths shall not exceed 20 feet without special safety provisions such as intermediate platforms and minimum 48-inch diameter risers.

The difference between the highest pipe invert entering a manhole and the invert leaving shall not exceed 24 inches. Manholes exceeding 24 inches of fall shall be designed as drop manholes.

5.3 MATERIALS AND INSTALLATION

This section describes standards for the construction of storm drain systems based on the most recent versions of all reference publications. The designer is responsible for procuring and complying with the most current version of each applicable reference document. For hydraulic design, the most restrictive criteria among said references and these Drainage Criteria are to be used.

5.3.1 Construction Materials

Storm drain pipes may be of any material specified in the City’s Standard Construction Specifications and as specified herein.

Pipe Material and Shape

All storm drain pipes shall comply with the City Standard Specifications for construction of underground utilities, as well as the most recent revision of the Wyoming Department of Transportation Standard Specifications for Road and Bridge Construction (WYDOT).

Public storm drain trunk mains may be circular, elliptical or arch pipe, or box culverts (rectangular – concrete only), or reinforced concrete pipe, smooth wall polyethylene, or polyvinyl chloride. However, pipe material and shape shall provide the ability to maintain full cross-sectional area and function without excessive cracking, breaking, or undergoing excessive deflection.

Corrugated metal pipe (CMP) is not permitted for storm drain construction, with the exception of slotted drain and private driveway culverts. Other pipe materials may be used under driveways, and through drainageways and open space areas. All other pipe materials shall be subject to City approval.

Corrosion, abrasion, and other appropriate observations of field conditions shall also be considered in determining appropriate pipe materials. Corrosion resistance shall be evaluated based on minimum resistivity, pH, sulfate content, and chlorine content of the soil and groundwater. Tests shall be conducted along the proposed alignment of the drainage system. Pipe wall strengths and coatings shall be suitable for soil conditions, design depths, and trench details.

All storm drain improvements placed under traffic carrying routes shall be designed to withstand static and live loads in accordance with the pipe manufacture's recommendations. As a minimum, all storm drains shall be designed to withstand an AASHTO HS-20 loading.

5.3.2 Pipe Size and Bury Depth

Where minimum buried depth cannot be met, the pipe design may be approved by following the variance procedure and submitting supporting documentation. Supporting documentation shall include pipe strength calculations, loading conditions, soil conditions, trench cross-sections, bedding materials, and any other information necessary to determine the suitability of the proposed design.

5.3.3 Manholes

Manholes and junction boxes shall be pre-cast with cast-in-place or pre-cast bases and eccentric cones and iron rings and covers. Flat top lids may be used if design depths are too shallow to allow eccentric cones, but shall not intrude into the pavement section. Steps shall be installed at appropriate spacing in line with the eccentric cone to provide safe access to the storm drain. Adjustments to rings and covers to match design grades shall be made with pre-cast concrete or HDPE rings and shall not exceed 8 inches in height.

Manholes exceeding 24 inches of fall shall be designed as drop manholes. Drop manholes with drop heights exceeding 6 feet shall be designed with high strength (6,000 psi) concrete. Non-standard manhole designs shall be subject to approval by the City Engineer.

5.4 MAINTENANCE AND ACCESS EASEMENTS

Storm drains shall be placed within street sections and easements to provide maintenance access.

For access to drainage facilities to be maintained by the City outside of already established public right-of-way, maintenance easements twice the pipe invert depth (at its deepest point) or 20 feet, whichever is greater, shall be conveyed to the City.

If required for drainage facilities, the minimum rear lot line easements shall be 10 feet on each lot or tract.

Right-of-way conveyed to the City shall provide the legal right-of-access to maintain drainage facilities so that they function as intended and they shall be kept clear of obstructions to flow and obstructions to access. Public right-of-way adjacent to drainage improvements may be used for easement requirements to provide maintenance access, upon approval of the City Engineer.

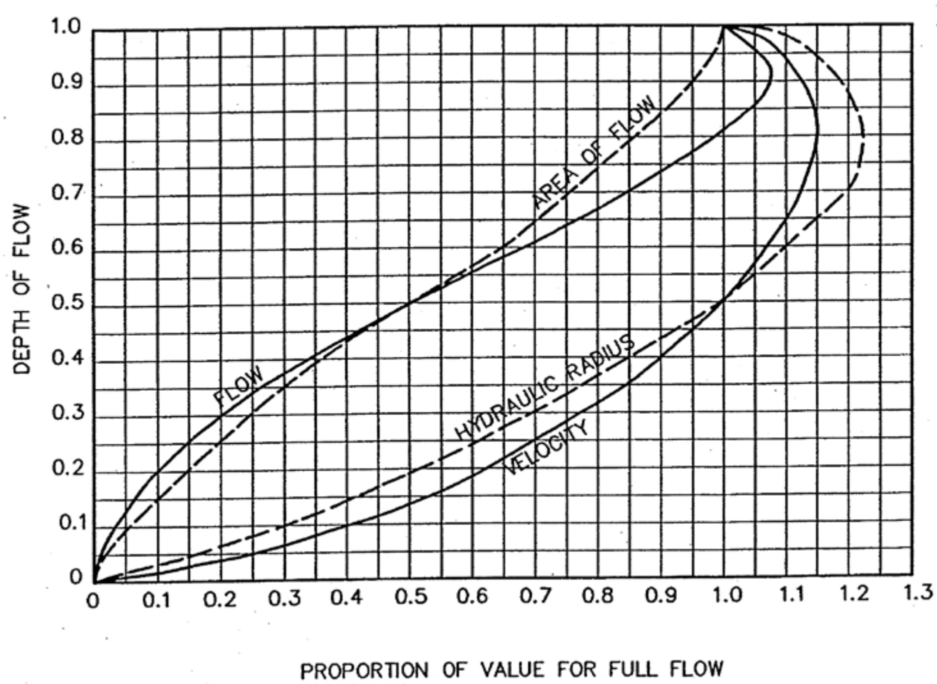
5.5 DESIGN EXAMPLE

Determine the depth of flow, *y*, flow area, and flow velocity in a storm drain (D = 2.75 ft, n = 0.013, and S_o = 0.003) for a flow rate of 26.5 cfs.

Just full flow conditions are computed first. From Equation 5.1, A_f = 5.94 ft^2, R_f = 0.69 ft, and Q_f = 29.1 cfs. Then, $V_f = Q_f/A_f$ = 29.1/5.94 = 4.90 ft/sec.

Now, by using Figure 5-2 with Q/Q_f = 26.5/29.1 = 0.91, it is determined that y/D = 0.73, A/A_f = 0.79, and V/V_f = 1.13. Therefore, y = (0.73)(2.75) = 2.0 ft, A = (0.79)(5.94) = 4.69 ft^2, and V = (1.13)(4.90) = 5.54 ft/sec.

Figure 5-2
Hydraulic Properties of Circular Pipe



5.6 CHECKLIST

All of the design criteria in this section must be followed. Several key considerations that the designer must take care to address include:

1. Design the EGL below the ground surface for the design event.
2. Design the HGL not to exceed the pipe's crown for the minor storm.
3. Design the HGL not to exceed 1 foot below ground when the conduit is designed to convey the major event.
4. Account for all losses in the EGL and HGL calculations including outlet, form, bend, manhole, and junction losses.
5. Provide adequate erosion protection at the outlet of all drains.
6. Provide cross-sections for riprap protection.
7. Check for minimum pipe cover and clearance with utilities.
8. Check overflow under sump conditions.
9. Design the invert of the inflow pipe to the detention basin to be higher than the water quality level.

SECTION SIX
INLETS

6.1 INTRODUCTION

This section presents criteria for selection and placement of storm drain inlets to conform to maximum allowable street ponding and flow spread requirements. All design submittals involving use of public streets for the conveyance of storm flows shall be reviewed based upon the criteria in this section as well as Section 5, *Storm Drains* and Section 7, *Streets*.

6.1.1 Inlet Types

Wherever street and gutter capacity exceeds allowable values based on flow spread, velocity, or depth, some or all of the runoff must be intercepted and diverted to an alternative conveyance facility, such as a storm drain or designated runoff channel. The most common method of interception is through the use of storm drainage inlets, of which there are three allowable types:

- Grate
- Curb Opening
- Combination (Curb Opening with Grate)

These three allowable inlet types are shown in Figure 6-1. All inlets must be labeled with "NO DUMPING – DRAINS TO CREEK." Inlet details are provided in the City's Design Standards.

6.1.2 Standard Inlets

Inlet types and applicable installations permitted for use in the City are listed in Table 6-1.

Table 6-1
Inlet Types and Applicable Installation

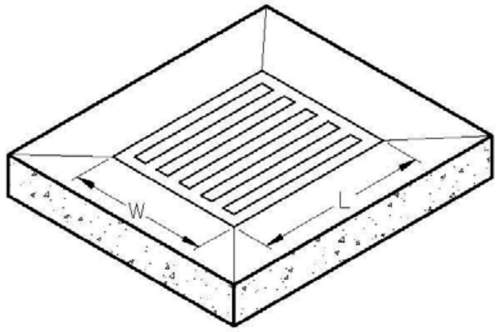
Inlet Type	Applicable Installation	Standard Detail
Single (Combination)	On-Grade / Sump	02725-01
Double (Combination)	On-Grade / Sump	02725-02
Triple (Combination)	On-Grade / Sump	02725-02
Curb Opening Inlet	On-Grade / Sump	02725-03
Area Inlet	Parking Lots / Open Area	02725-05

6.1.3 Inlet Grates

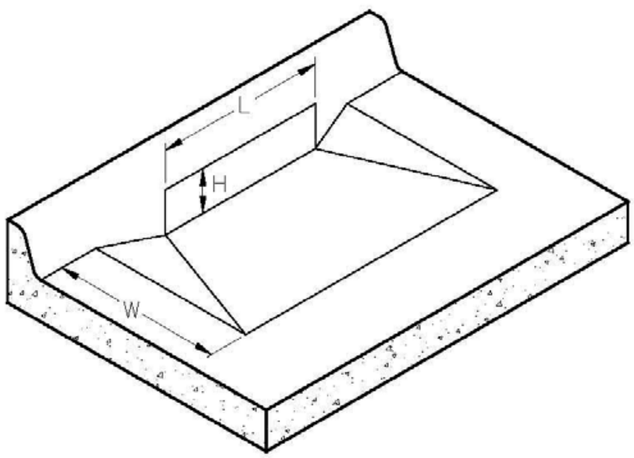
The City Design Standards include inlet grate types that are approved for use on streets, parking lots, and open sump areas. The primary types are Neenah R-3246, R-3295, and Deeter 2047. Two important notes concerning the selection of grates are:

1. Use Type A grates with combination inlets where inlet is located in sump condition.
2. Use Vane grates (Type L) where gutter flow is from one direction only.

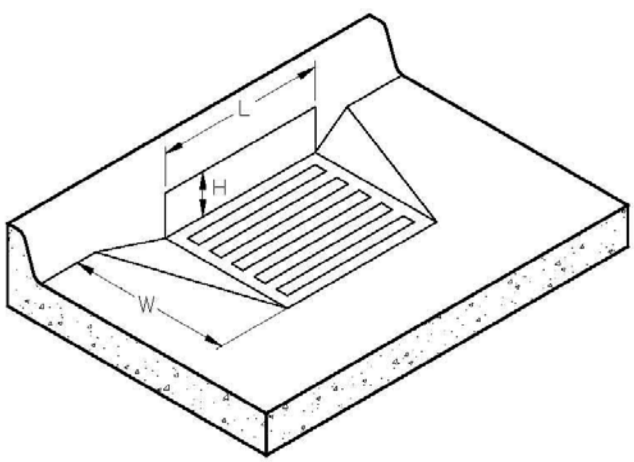
Figure 6-1
Inlet Types Isometric Views (HEC-22, FHWA)



a. Area Inlet (Grate)



b. Curb Opening Inlet



c. Combination Inlet

6.2 INLET DESIGN

Inlets may be located on a continuous grade, where flow not intercepted by the inlet will pass to the next location, or in a sag portion of a street's vertical alignment (or any other sump location). Inlets should generally be located upstream rather than downstream of pedestrian curb ramps and driveways. Curb opening inlets are preferred unless utilities are present, in which case combination inlets are allowed. A 2-foot concrete apron shall be provided around area inlets when no curb and gutter is present. Computation of inlet capacity involves type of inlet, location (on-grade or in a sump), grate type (if applicable), inlet geometry, approach flow width and depth, and longitudinal and cross slopes.

Methods and rationale to be utilized for the calculation of inlet capacities are those presented in the *Urban Drainage Design Manual, HEC-22* (FHWA 2001). However, HEC-22 contains a nomograph for finding grate splash-over velocity, and these Drainage Criteria use an empirical formula for this value (Guo 1999a). Additionally, clogging must be considered in the selection and location of inlets when inlet clogging is expected.

6.2.1 Inlet Clogging

Inlets are always susceptible to clogging when water flows to them; therefore, the City requires the use of clogging factors defined in this section for use in inlet design. Low flows can carry leaves and finer sediments and deposit them at the inlet, while higher flows can deposit larger debris. The latter is of most concern since it is often too large to be washed down the inlet, even with heavy storm flows. Proper inlet design and timely maintenance including street sweeping, visual inspection of inlets, and removal of large debris are essential to the proper operation of inlets.

Table 6-2 lists clogging factors for different inlet types, which are based on the assumption that routine maintenance is performed on all drainage structures.

Table 6-2
Clogging Factors for Single Storm Inlets

Condition	Inlet Type	Clogging Factor, C_1
Sump	Curb Opening	0.2
Sump	Combination	0.35
Continuous Grade	Curb Opening	0.2
Continuous Grade	Combination	Same as percentage for type of grate times 1.10
Sump	Area Inlet	50

In the case of multiple (double or triple) inlets, the continuous (linear) application of some of these factors would result in the inability to capture 100% of the flow incident to the subject inlet. Although on-grade inlets are rarely designed to capture 100% of gutter flow from the minor storm event, inlet design lengths may become unnecessarily long to achieve the desired flow interception rate. It is reasonable to assume that a majority of the sediment and debris is deposited at the inlet by the first few minutes of significant storm flow, possibly before the peak flow occurs at that inlet. Also, the majority of clogging tends to more greatly affect the upgradient segments of a multi-unit inlet. For instance, the first grate of a triple inlet may become 50% clogged, while the third grate experiences only 10% clogging. This phenomenon was generalized by Guo in *Design of Grate Inlets with a Clogging Factor* (2000) using a decay equation and empirically derived decay factors. Equation 6.1 should be applied to inlets on a continuous grade and inlets in sump conditions when inlet clogging is expected. This equation indicates that the clogging factor, C, is dependent only on the number of units.

$$C = C_1 \left(1 + e + e^2 + \ldots + e^{N-1}\right)/N \qquad (6.1)$$

Where:

C = Multiple unit clogging factor
C_1 = Single unit clogging factor (see Table 6-2)
N = Number of units
e = Decay ratio (0.5 for grates; 0.25 for curb opening inlets)

On-Grade Inlets

To apply the clogging factor to an on-grade inlet, the designer must first find the effective length (L_e), which is that portion of the inlet length that is considered unclogged.

$$L_e = (1 - C)*NL \qquad (6.2)$$

Where:

L = Measured length of the unit inlet opening (ft)

Sump & Area Inlets

Similarly, when orifice flow occurs at sump and area inlets, a clogging factor shall be applied to the opening area, A_o, of an inlet as:

$$A_{oe} = (1 - C)*A_o \qquad (6.3)$$

Where:

A_{oe} = Effective area of an inlet in a sump (sf)

The adjusted value is used in the computation of gutter flow parameters such as splash-over velocity, side-flow interception ratio, and curb-opening efficiency.

6.2.2 Inlet Interception Efficiency and Capacity

A properly designed inlet must have the capacity to effectively capture all of the runoff ponding in the gutter and sump to maintain acceptable ponding depths. Generally, the acceptable maximum ponding depth is the curb height. Inlet interception efficiency, E, is defined as:

$$E = Q_i/Q \qquad (6.4)$$

Where:

Q_i = Intercepted flow rate (cfs)
Q = Gutter total flow rate (cfs)

Flow that is not intercepted by the inlet is called bypass flow, Q_b:

$$Q_b = Q - Q_i \tag{6.5}$$

Unlike inlets on a continuous grade, sump and area inlets must provide 100% efficiency to intercept the design inflow with no bypass flow. Equations for estimating inlet efficiency used by the City are shown in Table 6-3. These inlet interception design efficiencies must consider the additional reduction due to inlet clogging explained in Section 6.2.1. The methodology for estimating inlet efficiency for each type of inlet is further described in the following paragraphs.

Table 6-3
Inlet Types and Design Efficiency[1], E.

Installation	Inlet Type	Design Efficiency [2]	Evaluation Note
On-Grade	Combination Inlet	$E=R_fE_0+R_s(1-E_o)$	Consider Grate Capacity Only
	Curb Opening Inlet	$E=1-[1-(L_e/L_T)]^{1.8}$	$L < L_T$, otherwise E = 1.0
Sump	Combination Inlet	$E=Q_{Gw}/Q$	The grate operating as a weir.
		$E=(Q_{Go}+Q_{Cw})/Q$	The grate operating as an orifice, & the curb opening operating as a weir.
		$E=(Q_{Go}+Q_{Co})/Q$	Both the grate and curb opening operating as an orifice.
	Curb Opening Inlet Only	$E=Q_{Cw}/Q$	The curb opening operating as a weir.
		$E=Q_{Co}/Q$	The curb opening operating as an orifice.
Parking Space / Open Area	Area Inlet	$E=Q_{Gw}/Q$	The grate operating as a weir.
		$E=Q_{Go}/Q$	The grate operating as an orifice.

Notes:

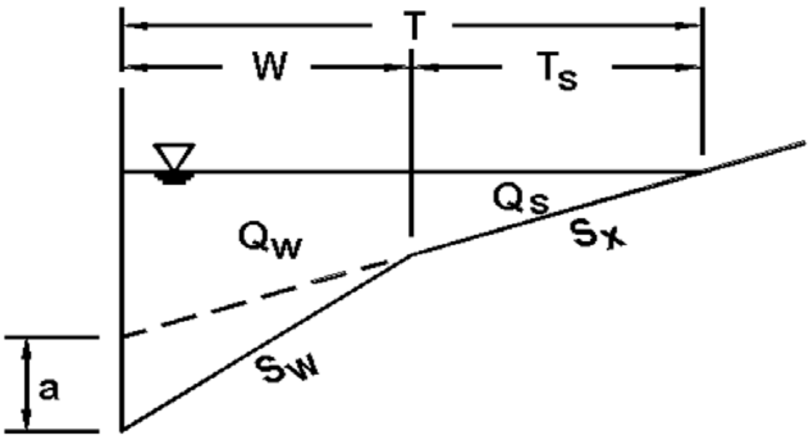
1. All design capacity needs to consider the clogging reduction described in Section 6.2.1.
2. Where E_o=the ratio of frontal flow to total gutter flow (Eq. 6.7); R_f= the ratio of the intercepted frontal flow to total frontal flow (Eq. 6.10); R_s= the ratio of the intercepted side flow to total side flow (Eq. 6.12); L_e= effective curb-opening length (Eq. 6.2); L_T= curb opening length required to intercept 100% of gutter flow (Eq. 6.15); Q_{Gw}= the intercepted grate flow operating as weir flow (Eq. 6.18); Q_{Go}= the intercepted grate flow operating as orifice flow (Eq. 6.20); Q_{Cw}= the intercepted curb-opening flow operating as weir flow (Eq. 6.18); and Q_{Co}= the intercepted curb-opening flow operating as orifice flow (Eq. 6.20).

6.2.3 Continuous Grade Condition

The total rate of intercepted flow by a grate inlet (or standard combination inlet) on a continuous grade is the sum of the intercepted frontal flow and the intercepted side flow adjusted for the inlet clogging due to the buildup of debris and sediment, which can adversely affect the interception capacity of an inlet. The flow along a continuous grade is primarily in one direction on the longitudinal street slope. Frontal flow, Q_w, is defined as that portion of the flow within the width of the grate, and the portion of the flow outside the grate width is called side flow, Q_s, as illustrated in Figure 6-2.

Gutter velocities exceeding a "splash-over" velocity and flow widths greater than grate widths typically result in interception efficiencies of less than 100% for an on-grade inlet.

Figure 6-2
Typical Composite Gutter Section (HEC-22, FHWA)



Frontal flow, $Q_{w,}$ is calculated using the following equations (HEC-22, 2001):

$$Q_w = QE_o \quad (6.6)$$

Where:

Q_w = Total frontal flow (cfs)
Q = Total gutter flow (no behind curb flow) (cfs)
E_o = Ratio of frontal flow to total gutter flow, determined using the following equation:

$$E_o = 1/ \{1 + S_w / S_x / [1 + (S_w / S_x / (T/W – 1))^{8/3} – 1]\} \quad (6.7)$$

Total side flow, Q_s, is the remainder of total gutter flow between the edge of the grate(s) and the street centerline:

$$Q_s = Q – Q_w \quad (6.8)$$

To find the ratio (R_f) of intercepted frontal flow (Q_{wi}) to total frontal flow (Q_w), the designer must first find the "splash-over velocity", V_o, for the selected grate type. Splash-over velocity is an experimentally derived value at which frontal flow begins to bypass the grate essentially because water does not spend enough time over the inlet to allow it to fall through the grate. Splash-over velocity varies with grate type, and some grates tend to better capture higher velocity flows. This function was developed empirically by Guo (1999a):

$$V_o = \alpha + \beta L_e – \gamma L_e^2 + \eta L_e^3 \quad (6.9)$$

Where:

L_e = Effective length (see Equation 6.2) (ft)
$\alpha, \beta, \gamma, \eta$ = Splash velocity constants (see Table 6-4)

The designer can then use the following equation to calculate the frontal flow interception ratio:

$$R_f = Q_{wi}/Q_w = 1.0 - 0.09(V-V_o) \quad (6.10)$$

Where: $V \geq V_o$, otherwise $R_f = 1.0$

The velocity can be estimated by:

$$V = Q/A \quad (6.11)$$

Where:

A = Area of flow ($A = 0.5[T^2S_x + W^2(S_w - S_x)]$) (for Type "A" High Back curb) (sf)

Table 6-4
Splash-over Velocity Constants

Type of Grate	Assumed Equivalent	α	β	γ	η
Bar P-1-7/8	Type A	2.22	4.03	0.65	0.06
Vane Grate	Vane	0.30	4.85	1.31	0.15

Source: Adapted from Guo, *Storm Water System Design*, 1999a.

Due to the primarily longitudinally flowing nature of street and gutter flow, the interception ratio of side flow is typically minimal, and in some circumstances is ignored altogether. This ratio, R_s, is defined (HEC-22, 2001) as:

$$R_s = Q_{si}/Q_s = 1 / [1 + 0.15V^{1.8}/(S_xL_e^{2.3})] \quad (6.12)$$

Where:

Q_{si} = Intercepted side flow (cfs)
V = Gutter flow velocity (fps)
S_x = Street cross slope
L_e = Effective length (ft)

Total intercepted flow (Q_i), then, is found using Equation 6.13:

$$Q_i = Q_{wi} + Q_{si} = R_fQ_w + R_sQ_s$$
(6.13)

Total capture efficiency (E) for a grate inlet can be found using Equation 6.14:

$$E = R_f\,(Q_w/Q) + R_s\,(Q_s\,/Q) = R_f\,E_0 + R_s\,(1-E_o) \quad (6.14)$$

Selection of grates for on-grade and sump inlets other than City standards must be consistent with this section. The designer shall obtain any grate rating data available from the manufacturer of the selected grate. This data may be used to cross-check the values obtained using the methods presented in these Drainage Criteria and/or to obtain interception and capacity values. However, in the latter case, the designer must incorporate all clogging/safety factors that apply to the inlet design per City requirements.

Curb Opening Inlets

The interception capacity of a curb-opening inlet on a continuous grade is dependent on inlet length, flow depth, and the longitudinal and cross slopes of the street. To determine the capture efficiency of a curb-opening inlet, one must first calculate the length, L_T, which would be required for 100% interception of the gutter flow at that location:

$$L_T = 0.6Q^{0.42}S_L^{0.3}\,[(1/\,(nS_e)]^{0.6} \qquad (6.15)$$

$$S_e = S_x + (a/W)\,E_o \qquad (6.16)$$

Where:

Q = Total gutter flow (cfs)
S_L = Longitudinal street slope
n = Manning's roughness coefficient
S_e = Equivalent cross slope
S_x = Street cross slope
a = Gutter depression below street slope (ft)
W = Gutter width (ft)

The efficiency (E) of a curb-opening inlet is calculated using:

$$E = 1 - [1 - (L_e/L_T)]^{1.8}$$
(6.17)

Where:

$L_e < L_T$, otherwise $E = 1.0$
L_e = Effective length (ft)

Combination Inlets

The interception capacity of a combination inlet is no greater than that of the grate(s) acting alone. However, the designer shall note the reduced clogging susceptibility of the combination inlet when compared with a grate-only configuration.

6.2.4 Sump and Area Inlet Condition

A sag or sump condition occurs in a location where water that flows into the area must pond to some depth before any of the flow can escape the area by a channel or overland flow. Unlike inlets on a continuous grade, sump inlets must have the capacity to capture effectively all of the runoff that ponds in the sump and to maintain acceptable ponding depths.

Because of this and an increased potential for inlet clogging due to low flow velocities, special provisions for the design of sump inlets must be made. A secondary flow path must be provided to maintain a reasonable ponding depth in the case of inlet failure (e.g., near-complete clogging). The preferred secondary flow path is a designated emergency overflow channel located within an accessible drainage easement. If no easement or channel is available at the inlet location, flanker inlets must be installed in the same gutter on each side of the primary inlet. Flanker inlets are located upgradient 10 to 50 feet from the primary sump inlet. The two flanker inlets shall have a combined design capacity equal to or greater than that of the primary inlet, and shall be located such that the maximum allowable ponding depth is not exceeded for the design storm.

The City requires the use of combination inlets in sumps due to their higher capacity and lower clogging tendency. Curb-opening inlets are also allowable, but grate-only inlets are not allowed for use on city streets.

The hydraulic capacity of an inlet in a sump condition is dependent on the configuration of the inlet and depth of ponded water. At small depths, the inlet operates by weir flow, transitioning to orifice flow at increasing flow depths. Weir and orifice depths are defined in Table 6-5.

The gross capacity of an inlet operating as a *weir* is defined by:

$$Q_{Gw} = C_w L_{we} d^{1.5} \quad \text{or} \quad Q_{Cw} = C_w L_{we} d^{1.5} \tag{6.18}$$

Where:

Q_{Gw} = Grate inlet weir capacity (cfs)
Q_{Cw} = Curb inlet weir capacity (cfs)
d = Flow or ponding depth (ft)
C_w = Weir discharge coefficient (see Table 6-5)
L_{we} = Effective weir length (ft) and is estimated using

$$L_{we} = (1 - C)*L_w \tag{6.19}$$

Where:

L_w= the length of perimeter of the grater receiving flow or length of curb opening.

The gross capacity of an inlet operating as an *orifice* is defined by:

$$Q_{Go} = C_o A_{oe}(2gd_o)^{0.5} \quad \text{or} \quad Q_{Co} = C_o A_{oe}(2gd_o)^{0.5} \tag{6.20}$$

Where:

Q_{Go} = Grate inlet orifice capacity (cfs)
Q_{Co} = Curb inlet orifice capacity (cfs)
A_{oe} = Effective Orifice open area (sf)
g = Gravitational acceleration (32.2 ft/s^2)
d_o = Depth to orifice centroid (ft)
C_o = Orifice discharge coefficient (see Table 6-5)

Table 6-5
Discharge Coefficients and Variable Definitions for Inlets in a Sump Condition

Type of Inlet	C_w	C_o	Weir	Orifice
Grate	3.00	0.67	$d < 1.79\ (A_{oe}/L_{we})$	$d > 1.79\ (A_{oe}/L_{we})$
Curb Opening	3.00	0.67	$d < h$	$d > 1.4h$
Depressed Curb Opening	2.30	0.67	$d < h + a$	$d > 1.4h$
Area Inlet	3.00	0.67	$d < 1.79\ (A_{oe}/L_{we})$	$d > 1.79\ (A_{oe}/L_{we})$

Notes:
a = Depth of depression
h = Curb Height

It is important to note that the capacity of a combination inlet in a sump is defined by the capacity of the grate portion only when operating as a weir (the curb opening is ineffectual and thus ignored), but is defined by the cumulative capacity of the grate and curb opening when operating as an orifice. Any curb opening length extending beyond the ends of the grates may be included in the weir length.

For any given inlet, a certain range of depths will result in "transitional flow," where neither the weir nor orifice equation accurately models flow through the inlet. Linsley (1992) states that where transition conditions exist, "the capacity is intermediate between that of an orifice and a weir." For design purposes, the capacity for depths in the transitional range is based on the lesser of the results of the weir and orifice equations.

Local inlet depression increases the capacity of inlets, especially those in sumps, by increasing the depth over the inlet without increasing street flow depth. Local depression loses effectiveness for curb-opening inlets of 12-foot length or greater, so the "Curb-Opening" information from Table 6-5 shall be used for these.

6.3 INLET SPACING

Storm inlets shall be placed at low points in flow lines and sumps, median breaks with catch curbs and where necessary to maintain allowable street flow depths for Minor and Major storm events. No more than two inlets shall be connected by each lateral pipe entering trunk line manholes or junction boxes.

Inlets shall be placed at any location where ponding water may encroach on street traffic beyond the allowable limits. These limits are defined by gutter flow depth during the Minor storm event (see Section 7, *Streets*). An inlet location is determined using an iterative process:

1. Determine a preliminary location for the inlet based on street configuration and estimated runoff to the gutter.
2. If the inlet is in a sump, location is essentially fixed during the remainder of the design process. The inlet shall be sized to maintain ponding depths smaller than those required by the City of Gillette. If the required inlet size becomes excessively large, the designer is urged to install additional inlets upgradient from the sump.
3. For inlets on a grade, the designer must find the flow characteristics at the selected preliminary inlet location to determine whether the inlet needs to be placed further upstream or may be moved downstream based on maximum flow depth for the Minor storm event.
4. The designer shall take into account the change in tributary area to the inlet associated with any upstream or downstream movement.
5. A typical design interception efficiency of an on-grade inlet is 70 to 80 percent. As mentioned previously, on-grade inlets designed to capture 100 percent of the minor storm runoff tend to be significantly less effective both hydraulically and economically.
6. The designer shall include any carryover (bypass) flow from an upstream inlet when calculating the flow at a downstream inlet. Although the peak runoff to an inlet may not coincide with the peak carryover flow from an upstream inlet, these two peak flows shall be added to find the total peak flow to the downstream inlet.

Maximizing the use of sump inlets tends to increase the overall efficiency of the inlet system, and inlets must be installed at all street sags (vertical curve low points) and at all sumps formed by intersections except where other drainage provisions have been made. Therefore, it is suggested that sump inlets are located prior to placement of any on-grade inlets during the design process.

Maintenance

Locations where significant sediment deposits may occur should be identified so that maintenance operations can include inlet cleanout as necessary. Inlets shall be designed to function properly based on expected sediment and debris clogging as specified in Section 6.2.3, Inlet Clogging.

6.4 CHECKLIST

All of the design criteria in this chapter must be followed; however, key considerations that the designer must address include:

1. Inlets are required at sumps, median breaks with catch curbs, locations where allowable street capacity has been exceeded, and intersections.
2. Inlets should generally be located upstream rather than downstream of pedestrian curb ramps and driveways.
3. Inlets should be spaced in a manner to prevent clogging. This is particularly critical for flat grades and sump conditions.
4. Curb opening inlets are preferred unless utilities are present, in which case combination inlets are allowed.
5. A 2-foot concrete apron shall be provided around area inlets when no curb and gutter is present.
6. An emergency overflow route must be provided in sump areas.

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SECTION SEVEN
STREETS

7.1 INTRODUCTION

This section presents criteria for design of public streets for stormwater drainage and allowable encroachment of these flows in public street rights-of-way. All design submittals involving the utilization of public streets for the conveyance of storm flows shall be reviewed based upon the criteria in this section as well as Section 5, *Storm Drains* and Section 6, *Inlets*.

7.1.1 Function of Streets in the Drainage System

The primary function of public streets is the movement of traffic. Therefore, use of streets as part of the drainage system must be limited in order to minimize interference with traffic functions. Streets typically convey runoff collected on the street surface and from some portion of the surrounding areas. As an integral part of the drainage system, streets must be capable of conveying stormwater runoff to a primary drainage conveyance facility such as a storm drain system or open channel. Allowable street capacity is based on its cross-section, longitudinal slope, and the maximum allowable ponding depth. Where the Minor storm event flow depth exceeds the maximum allowable depth, inlets must be used to collect runoff and reduce street flow. During a Major storm event, streets become emergency runoff channels, conveying floodwaters away from structures as much as possible. During a Major event, many streets can be inundated to a degree where they become impassable to vehicles and dangerous to pedestrians.

7.1.2 Drainage Impacts on Traffic and Streets

The following drainage conditions affect streets and traffic movement.

Sheet Flow

Rainfall on the paved surface of a street flows overland (sheet flow) according to the cross slope until it reaches a curb-and-gutter section or roadside ditch. Generally, the depth of sheet flow is zero at the street crown and increases towards the collection channel at the edge of the pavement (e.g., the gutter). Sheet flow depths are also dependent on longitudinal slope.

Sheet flow can interfere with traffic movement by increasing the risk of hydroplaning or splashing. Hydroplaning is a phenomenon in which one or more of a vehicle's tires lose significant contact with the pavement and become supported by a thin layer of water, which can cause loss of vehicle control. The potential for hydroplaning increases with vehicle speed and with the depth of water on the road surface. Splashing is also dependent on vehicle speeds and water depth, and can interfere with traffic movement by reducing driver visibility.

In wide arterial or highway sections, increasing street cross-slope may help to reduce sheet flow depths and splashing potential in the outside lanes. In general, a 2% cross slope is desirable to promote swift removal of runoff and reduce sheet flow depths while minimizing potential vehicle side-slippage from ice buildup during winter months.

Gutter Flow

Where curb-and-gutter is used, all runoff tributary to a street will be directed to, and flow in, the gutter until it reaches a storm drain inlet or other primary conveyance. Generally, flow width (spread) will increase in the downstream direction, eventually spreading into the traffic lane(s).

Flow spreads affect traffic movement. Where on-street parking is allowed (e.g., residential streets), flow spread to the inner edge of the parking lane will not interfere with moving traffic. However, where no street parking is allowed, flow spreads beyond the gutter will reach into a travel lane, potentially causing traffic disruption. Some interference with traffic movement is allowable and expected, but the degree of such interference is to be minimized. Emergency vehicles must be able to travel City streets without encountering extensive or impassable ponding depths.

Allowable limits on flow widths and depths for City streets are summarized in Section 7.2, Street Classifications and Capacity Limitations.

Intersections and Localized Cross Flow

Grade changes and street crowns at intersections can cause storm runoff to pond at depths greater than the intended design depth. This localized, temporary ponding poses a high safety risk, especially on roads with higher speed limits (e.g., arterials). Street and especially intersection design must incorporate measures to control the stormwater depths at locations where temporary ponding may occur through the use of grade changes or additional inlets.

Cross flow occurs at intersections where gutter flow from the cross street spills across the intersection. Where allowable, a concrete crosspan (valley gutter) may be used to direct these flows (see City Design Standards, 2009, Dwg. No. 02530-01). Localized cross flow can have the same negative effects on vehicular safety and traffic movement as sheet flow and temporary ponding. Therefore, design of the storm drain system must consider potential cross flow locations to minimize its effects.

Pavement Deterioration

Stormwater runoff affects roadway maintenance and repair and can contribute to the structural failure of the roadway pavement. The latter situation occurs when water penetrates the pavement and reaches the subgrade. Especially in climates subject to freezing and thawing, subgrade saturation and/or material washout will eventually cause subgrade failure, leading to pavement failure and an unsafe/unstable road surface.

Although undamaged pavement surfaces generally keep water from penetrating through to the subgrade, it is possible for some water to seep through with time. Storm drainage design for City streets shall incorporate measures to decrease frequency of pavement submergence to minimize seepage potential.

Sedimentation and Debris

Sediment and debris buildup occurs on streets in any area where flow velocities decrease, such as near grade changes and inlets. Sediment and debris affects the flow capacities of gutters and streets, and causes increased flow spread and interference with traffic movement. Locations where significant deposits may occur should be identified for maintenance purposes to include street sweeping and inlet cleanout as necessary. Inlets shall be designed to function properly based on expected sediment and debris clogging as specified in Section 6. Localized sedimentation issues due to construction activities shall be controlled per the criteria presented in Section 11, Erosion and Sediment Control, of these Drainage Criteria.

7.2 STREET CLASSIFICATIONS AND CAPACITY LIMITATIONS

Streets are classified according to estimated traffic volume and right-of-way width. The City has adopted Design Standards that include standard drawings and details for the construction of streets and location for utilities. These drawings and details were developed and are maintained by the Gillette Department of Engineering. Some of the street classifications specified therein have been grouped together in these Drainage Criteria due to similar hydraulic characteristics.

The Design Standards include two curb types, high back (Type A) and rollover (Type C). The high back curb configuration has a height of 6 inches. This configuration or an approved alternative design must be used for arterial and collector streets. The rollover curb configuration may be used only for local and local through streets. The total vertical height from the gutter flowline to the "top" of the curb is 4 inches, resulting in a lower maximum minor storm event water surface elevation for streets utilizing this gutter type.

Allowable use of streets for storm flows is summarized in Tables 7-1 through 7-3. The Minor storm referenced in these tables is either the 2-, 5- or 10-year event in accordance with Section 2, Table 2-1, and the Major storm is the 100-year event. No curb overtopping during the Minor storm is allowed for any street, regardless of classification.

Table 7-1
Allowable Use of Streets for Minor Storm Runoff

Street Classification	Maximum Flow Spread	Standard Drawing No.
Local, local through	No curb overtopping. Flow may spread to crown of street.	02512-02
Collector	No curb overtopping. Flow spread must leave at least one lane free of water, with 6 feet on either side of the street crown.	02512-01
Minor/Major Arterial	No curb overtopping. Flow spread must leave at least two 12-foot lanes free of water, providing 12 feet on each side of the street crown or median.	02512-01

Table 7-2
Allowable Use of Streets for Major Storm Runoff

Street Classification	Maximum Depth and Inundated Area
Local, local through and collector	Residential dwellings and public, commercial, and industrial buildings shall not be less than 12 inches above the 100-year water surface elevation at the ground line or lowest water entry into the building. The depth of water over the gutter flow line shall not exceed 12 inches.
Arterial	Residential dwellings and public, commercial, and industrial buildings shall not be less than 12 inches above the 100-year water surface elevation at the ground line or lowest water entry into the building. To allow for emergency vehicles, the depth of water shall not exceed the street crown or 12 inches at the gutter flow line, whichever is more restrictive.

Table 7-3
Allowable Cross-Street Flow When Cross Pans Are Allowed

Street Classification	Minor Storm Flow	Major Storm Flow
Local, local through	6 inches of depth in cross pan, if cross pan allowed.	12 inches of depth in cross pan or gutter flow line.

7.3 HYDRAULIC EVALUATION FOR STREET CAPACITY

Calculations for flow capacity and velocity in a given street section are based upon the maximum depths from the tables above and the assumption that any area not within the street right-of-way would not contribute to the capacity of the street system (called "ineffective flow area"). Therefore, for calculation purposes, it is assumed that an infinitely high vertical wall of zero roughness exists at the right-of-way boundary, and any flow area outside this boundary is not considered in analysis. Due to the potential for a single street cross-section to have different half-street cross-sections, all street capacity calculations are to be completed on a half-street basis. Therefore, the same vertical wall assumption applies to the street centerline as to the right-of-way where the calculated flow width exceeds the half-street width.

Gutter and street flow can generally be assumed to be uniform for the purpose of hydraulic evaluation and design, but as street flow depth increases, flow width increases at a much faster rate. This wide, relatively shallow flow has the effect of decreasing the hydraulic radius, rendering the standard Manning's equation somewhat inaccurate. The FHWA presents a modified form of the equation in *Introduction to Highway Hydraulics*, taken from HEC-22 (FHWA 2001):

$$Q_S = K_u S_x^{5/3} S_L^{1/2} T_S^{8/3} / n \qquad (7.1)$$

Where:

Q_S = Street flow capacity (not including gutter) (cfs)
n = Manning's roughness coefficient
K_u = 0.56 (English units)
S_x = Street cross slope (ft/ft)

S_L = Street longitudinal slope (ft/ft)
T_S = Flow top width (not including gutter) (ft)

For streets with a single cross slope for the gutter and street section, Equation 7.1 will suffice for determining total gutter/street flow capacity if the T_S term is replaced by T = Total Flow Top Width (including gutter, inside curb). However, the City Design Standards indicate composite cross slopes for all streets, and the gutter has a steeper cross slope than the street. For this case, Equation 7.1 specifies capacity in the flow area between the edge of pavement and the edge of flow, not including the gutter. Note that the result of the same equation as applied to any flow area beyond the street centerline must be subtracted from the total capacity. The total flow capacity in the street and the gutter is calculated as:

$$Q = Q_S / (1\text{-}E_o) - Q_{S,\ \text{Outside Street Centerline}} \tag{7.2}$$

$$E_o = 1/ \{1 + S_w/S_x/([1 + S_w/S_x /(T/W - 1)]^{8/3} - 1)\} \tag{7.3}$$

Where:

S_w = Gutter cross slope (ft/ft)
S_x = Street cross slope (ft/ft)
T = Top width (inside curb) (ft)
W = Gutter width (ft)

Where gutter flow does not encroach on the street surface, it is assumed that the width to depth ratio is not large enough to warrant the utilization of the HEC-22 modified Manning's equation. Instead, the standard form is used in these cases:

$$Q = 1.49/n\ AR^{2/3}S_L^{½} \tag{7.4}$$

Where:

Q = Flow capacity (cfs)
S_L = Street longitudinal slope (ft/ft)
R = Hydraulic radius (ft) = A/P
A = Cross-sectional flow area (ft^2)
P = Wetted perimeter (ft)

Major storm event flow depth may exceed curb height, thus the flow area behind the curb – between the curb and the right-of-way – must also be considered in the calculation of total street capacity. The flow in this area is found using Equation 7.1, replacing T_S with T_B = Flow Top Width (behind curb). Note that the truncation procedure described above must also be applied here if the calculated top width extends beyond the right-of-way boundary. Therefore, the total flow between the curb and street centerline, Q_B, is:

$$Q_B = Q_{B,\ \text{Gross}} - Q_{B,\ \text{Outside Street Right-of-Way}} \tag{7.5}$$

Total street capacity, then, is the sum of the resulting values from Equations 7.2 and 7.5. The HEC-22 procedure has certain limitations and includes certain assumptions, the most applicable of which are:

1. One value for Manning's roughness "n" must be used for the entire cross-section. This can be a composite value derived from multiple roughness segments in the cross-section.
2. The HEC-22 method ignores any roughness characteristics of the vertical segment of the curb. The energy-dissipating effect of this portion is considered to be negligible when compared to the wide bottom sections.
3. The "high back" curb (Type A) actually slopes back away from the street by 2 inches while rising 6 inches. This slope is ignored by the HEC-22 equations. The gutter width is assumed to include this extra 2 inches with a vertical segment of 6-inch height at this point.
4. The Design Standards indicate that the edge of the roadway pavement is to be ½ inch above the edge of the concrete gutter. The HEC-22 method does not account for this directly, and the hydraulic effects are assumed to be negligible, so this is ignored.

Computer programs can be used to model sections that are more complex than the HEC-22 method can accurately model.

Certain assumptions can be made to simplify the design process. However, it is the responsibility of the designer to ensure that each assumption is valid for a specific design.

1. A Manning's roughness value of n = 0.016 can be assumed for all flow surfaces encountered within the street right-of-way when using the HEC-22 method.
2. A street cross slope of 2 percent can be assumed for arterial and collector streets and 3 percent for local, local through streets (this might not be true on curved streets).
3. A gutter cross slope of 4.17 percent can be assumed for all gutters.

In cases where these assumptions may not be valid, such as designs incorporating the use of non-standard street sections, the designer shall utilize the equations presented above to determine allowable street capacity.

7.4 HYDRAULIC EVALUATION FOR RURAL STREETS (LOW-VOLUME STREETS WITHOUT CURB AND GUTTER)

7.4.1 Roadside Ditches

Roadside ditches shall be used in lieu of curb and gutter when rural street sections are approved. Maintenance shall be considered when designing and using roadside ditches, including adequate area and side slopes to allow for maintenance access and vehicles. Maximum side slopes of 4 (horizontal) to 1 (vertical) are required. Roadside ditches shall be included in the street right-of-way section.

7.4.2 Roadside Ditch Design Criteria

Allowable flow depths and roadway encroachment in the Minor and Major storm events for rural roadside ditches are similar to streets with curb and gutter. Tables 7-1 and 7-2 reference allowable flow depth based on the gutter flow line, and can be used for rural roadside ditches by applying the depth at the edge of pavement rather than gutter flowline. The spread of flow shall not extend outside the street right-of-way, and at least 12 inches of freeboard shall be provided above the major storm water surface elevation to the lowest point of entry at any existing adjacent structures.

Rural roadside ditches shall be designed in accordance with the criteria for minor drainageway grass-lined channels shown in Section 4, Open Channels. Grade-control structures are required to maintain velocities less than the maximum allowable, or riprap lining shall be provided in accordance with these Drainage Criteria.

There are cases when the roadside ditch criteria may need to be more stringent due to the function of the rural road. Even if a rural road has a low traffic volume, it may be important for emergency access to several properties and, therefore, require special design criteria. The City reserves the right for more stringent criteria for single point access roads.

See Section 8, Culverts, for design criteria pertaining to rural roadside ditch culverts.

7.5 ALLOWABLE CROSS-STREET FLOW CONDITIONS

Cross-street flow can occur in an urban drainage system when the runoff in a gutter spreads across the street crown to the opposite gutter, or when the flow in a drainageway exceeds the capacity of a roadway culvert and subsequently overtops the crown of the street. Criteria for the first condition are discussed in the following sections. Allowable overtopping at culvert crossings is limited by the criteria provided in Section 8, Culverts.

The cross-street flow may be caused by super-elevation of a curve, by the intersection of two streets, by exceeding the capacity of the higher gutter on a street with cross fall, or by street design that has not met the criteria provided herein. Cross pans are not allowed on collector or arterial streets or where a storm drain is available.

Allowable cross-street flow depths when the flow depth exceeds the street crown elevation are provided in Table 7-4. In the Minor storm event, cross-street flow is NOT allowed based on the allowable flow depth and encroachment criteria provided in Table 7-1. In the Major storm event, allowable cross-street flow is controlled by the depth limitation presented in Table 7-4. For example, if the maximum allowable gutter flow depth is 12 inches and the crown of the road is 7 inches above the flowline of the gutter, 5 inches (12 inches minus 7 inches) of cross-street flow is allowed during a Major storm event.

Table 7-4
Allowable Cross-Street Flow Due to Spread

Street Classification	Minor Storm Flow	Major Storm Flow
All	Not allowed	12 inches above gutter flow line.

7.6 DESIGN EXAMPLE – DETERMINATION OF STREET CAPACITY

Determine the discharge in a composite gutter section if the allowable spread is 9.0 feet, the gutter cross slope Sw is 0.045. The street's longitudinal slope is 0.01, the cross slope is 0.02, and the curb height is 6 inches.

Using Figure 7-1, W = 2 feet, Ts = 7 feet. Equation 7.1 can now be used to find the flow in the street section.

Qs = [(0.56)(0.02)$^{5/3}$(0.01)$^{1/2}$(7.0)$^{8/3}$]/(0.016) = 0.93 cfs

Now with Sw/Sx = 0.083/0.02 = 4.15, T/W = 9.0/2.0 = 4.5, and T/W - 1 = 3.5, by using Equation 7.3.

E_o = 1/ {1 + 4.1/([1 + 4.1/3.5]$^{8/3}$ – 1)} = 0.63

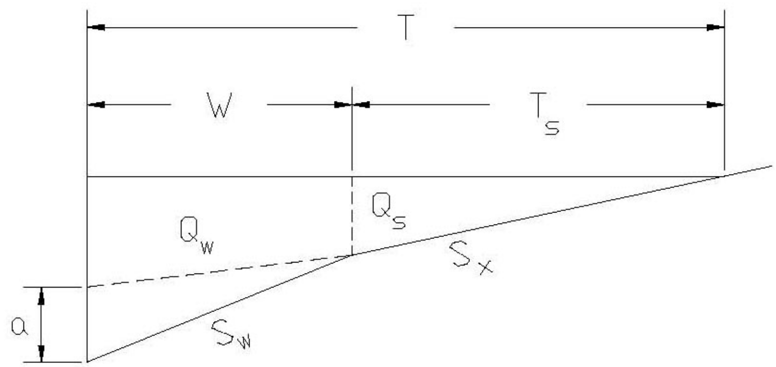
Now the theoretical flow rate can be found using Equation 7.2 as:

Q = [(0.92)/(1 - 0.63)] = 2.49 cfs

Then by using the relationship $y = a + TSx$, where $a = W(S_w - S_x)$, or y = 2 (0.083 – 0.02) + (9.0) (0.02) = 0.31 feet

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet).

Figure 7-1
Composite Gutter Cross-Section



7.7 CHECKLIST

All of the design criteria in this chapter must be followed. Several key considerations that the designer must take care to address include:

1. The primary function of urban streets is for safe traffic movement. Where a storm drain is available, inlets must be provided at intersections.
2. Provide an inlet where a catch curb changes to a spill curb.
3. Provide an inlet where allowable street capacity for Major and Minor storms is subject to safety considerations.
4. Nuisance flows must be carried by gutters or pans to an inlet. Nuisance flows are not allowed to cross a driving lane.

SECTION EIGHT
CULVERTS

8.1 INTRODUCTION

A culvert is a conduit for the conveyance of water under a roadway, railroad, or other embankment, and is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity. As distinguished from bridges, a culvert is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Generally, culverts are structures spanning 20 feet or less between the ends of openings for multiple boxes. However, a structure designed hydraulically as a culvert is treated in this Section, regardless of its span.

The size, alignment, and structural components of a culvert directly affect the capacity of the drainage system. A careful approach to culvert design is essential, whether in new development and retrofit situations, because culverts significantly influence upstream and downstream flood risks and public safety. Inadequate culvert capacity can force stormwater out of the conveyance system, overtop the embankment, and cause damage in and adjacent to the channel. In addition to the hydraulic function, culverts must carry construction, highway, railroad, or other traffic and earth loads. Therefore, culvert design involves both hydraulic and structural design considerations. The criteria for hydraulic design of culverts are presented in this section.

For structural considerations, the reader is referred to Section 5.1.2. A structural analysis demonstrating the structural adequacy of the pipe is required. This may be as simple as providing manufacturer's specifications that match site conditions, or it may require independent analysis. Buoyancy must be considered for culverts as well. For buoyancy, it is recommended that the structure is assumed to be blocked at the inlet, overtopping and full of air rather than water. The Concrete Pipe Handbook (ACPA 2007) presents an acceptable design procedure.

8.2 CULVERT HYDRAULICS

The general procedures for hydraulic evaluation and design of culverts can be found in the FHWA *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (HDS-5), (FHWA 2005). Research by the National Bureau of Standards sponsored and supported by the FHWA resulted in a series of reports providing a comprehensive analysis of culvert hydraulics under various flow conditions. These data were used to develop culvert design aids, called nomographs, which are the basis for the culvert design process. The more commonly used nomographs from HDS-5 are provided in Section 8.6, Design Aids. HDS-5 is the required method to be used in the hydraulic evaluation, sizing, and design of culverts in the City of Gillette, except as modified herein.

Flow in culverts is controlled by hydraulic conditions at the inlet, the outlet, and through the barrel. Under inlet control, the flow is controlled by the inlet geometry and the headwater driving the flow through the culvert. Under outlet control, the flow through the culvert is controlled primarily by culvert slope, roughness, and tailwater elevation. When designing a culvert, the designer must evaluate both inlet and outlet control conditions for the given design constraints (e.g., discharge, headwater, tailwater, and outlet velocity). The control condition which produces the greater energy loss for the design condition determines the appropriate control for design. Culvert hydraulic calculations shall be performed using standard rating nomographs, or culvert hydraulic analysis software which uses these procedures.

8.2.1 Design Criteria

The hydraulic design of a culvert essentially consists of an analysis of the required performance of the culvert to convey flow from one side of an embankment to the other. The designer must select a design flood frequency, estimate the design discharge for that frequency, and set an allowable headwater elevation based on the selected design flood and the following considerations. The actual design of a culvert installation is more difficult than the simple process of sizing culverts due to problems arising from topography and other considerations.

Discharge and Freeboard

Culverts placed where public roadway alignments cross drainageways shall be sized so that street overtopping is limited to depths shown in Table 8-1. For local, local through and collector streets, a dipped overflow section shall be allowed by the City only if the maximum velocity does not exceed 6 fps and the maximum depth does not exceed 6 inches at the street crown.

Table 8-1
Allowable Street Overtopping Depths at Culvert Crossings

Street Classification	Minor Storm	Major Storm
Local, local through	None	6 inches at street crown.
Collector	None	6 inches at street crown.
Arterial	None	No overtopping allowed.

In all cases 1 foot of freeboard must be maintained between the lowest point of the drive lane and the Minor storm water surface elevation. The low chord of curved pipes shall be at the lowest crown elevation, and the low chord for box culverts and bridges is the lowest point on a horizontal spanning member.

Headwater and Tailwater

Headwater (HW) is the upstream depth of water measured from the invert at the culvert entrance. In selecting the design headwater elevation, the designer should consider the following conditions:

- Anticipated upstream and downstream flood risks, for a range of return frequency events.
- Potential for damage to the culvert and the roadway.
- Potential for traffic interruption.
- Hazards to human life and safety.

- Roadway elevation above the structure and low point in the roadway profile grade line.
- Elevation at which water will flow to the next cross drainage.
- Elevation at which water will cause flow into adjacent properties.
- Relationship to stability of embankment that culvert passes through.
- Potential for upstream headcutting or erosion, siltation or clogging from debris.

The headwater elevation for the design discharge should be consistent with the freeboard and overtopping criteria in Section 4, Open Channels, of these Drainage Criteria. The designer should verify that the watershed divides are higher than the design headwater elevations. In flat terrain, drainage divides are often undefined or nonexistent and culverts should be located and designed for the least disruption of the existing flow patterns.

When the designer has addressed all these design considerations, the maximum HW for the 100-year design flow shall be 1.0, 1.2, or 1.5 times the culvert diameter or culvert rise dimension, as described below. If this value is higher than the elevations in the preceding design considerations, then the lowest elevation shall govern. Under high fills or in areas with flat ground slopes, a considerable volume of water may pond upstream of a culvert installation at the design HW depth, which may attenuate flood peaks under such conditions. However, culvert sizes may not be reduced based on this peak discharge attenuation. Following are recommended HW/D values for headwater depth:

- Culverts with cross sectional area, $A \leq 30\ ft^2$, $HW/D \leq 1.5$
- Culverts with cross sectional area, $A > 30\ ft^2$, $HW/D \leq 1.2$
- Culvers with a rise > 12 ft, $HW/D \leq 1.0$

Culverts must also be sized so that channel capacities and design water levels are maintained.

Tailwater is the depth of water downstream of the culvert measured from the outlet invert. Tailwater is based on the hydraulics of the downstream channel. Backwater calculations per Section 4 of this criteria from the downstream control point are required to define tailwater. In many circumstances when the downstream conditions are well known, normal depth approximations may be used instead of backwater calculations to determine tailwater.

Outlet Velocity

Since a culvert usually constricts the available channel area and often has smoother hydraulic surfaces, flow velocities in the culvert are likely to be higher than in the channel. Increased velocities at the culvert outlet can cause streambed scour and bank erosion. Minor problems can occasionally be avoided by increasing the barrel roughness.

Culvert design shall include an analysis of the channel stability at the outlet, which is governed by a maximum permissible mean velocity. This design method assumes that a given channel section will remain stable up to the maximum permissible velocity listed in Table 4-2 in Section 4, Open Channels. For unlined channels, velocities exceeding the values shown shall require outlet protection. Culvert outlet velocities shall not exceed the values given in Table 4-2 for lined channels.

Section 4.5.3, Low-Flow Grade-Control Structures, provides requirements for culvert outlet erosion protection. When riprap is not an option or is unavailable, the designer shall use other natural and man-made materials, such as gabions, concrete, recycled concrete, shotcrete, masonry, geotextiles, and woody plants. Outlet protection measures other than riprap may be used if approved by the City Engineer.

The outlet velocity is determined using Manning's Equation as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity.
2. If the controlling headwater is in outlet control, determine the area of flow at the outlet based on the barrel geometry and the following:
    a. Critical depth if the tailwater is below critical depth
    b. Tailwater depth if the tailwater is between critical depth and the top of the barrel
    c. Height of the barrel if the tailwater is above the top of the barrel

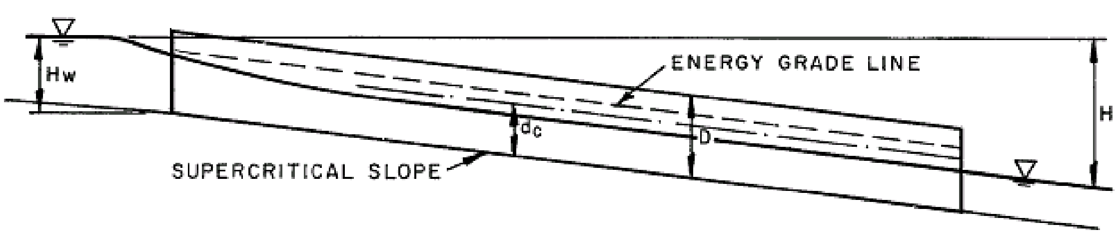
All culverts shall be designed to maintain a minimum velocity of 2 fps during the Minor storm event and to limit erosion potential during Major storm events, unless adequate erosion control or energy dissipation is provided at the culvert outlet.

8.2.2 Inlet Control

Inlet control occurs when the culvert barrel can convey more flow than the inlet will accept. The flow rate is controlled at the culvert entrance by the headwater depth, cross-sectional area, inlet edge configuration, and/or barrel shape. Under inlet control, the culvert barrel usually flows partially full. The control section of a culvert operating under inlet control is located just inside the entrance. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity.

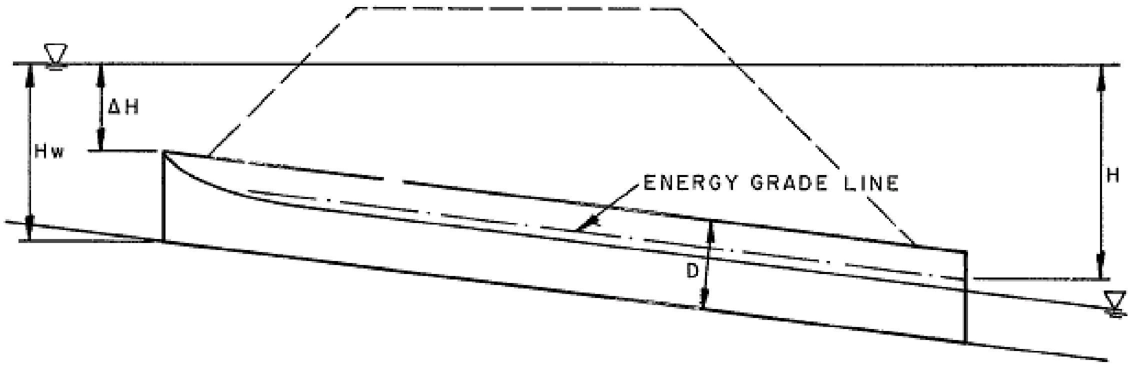
Inlet control for culverts may be an unsubmerged or submerged condition. In an unsubmerged condition, the headwater is not sufficient to submerge the top of the culvert as, shown in Figure 8-1. In this situation, the culvert inlet acts like a weir.

Figure 8-1
Unsubmerged Inlet



In a submerged condition, the headwater submerges the top of the culvert but the pipe does not flow full, shown in Figure 8-2. In this situation, the culvert inlet acts like an orifice.

Figure 8-2
Inlet Control – Submerged Inlet



For a culvert operating with inlet control, the upstream water surface elevation and the inlet geometry represent the major flow controls. Roughness, slope, length of the culvert barrel, and outlet conditions, including tailwater, are not factors in determining culvert hydraulic performance.

Inlet Control Calculation

Inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration in inlet control. The approach velocity head may be included as part of the headwater. Inlet control nomographs are used in the design process. Inlet control nomographs for typical configurations are included at the end of this section, Figures 8-6 through 8-8. For all other situations, refer to HDS-5 (FHWA 2005).

Use of inlet control nomographs is described in the following paragraphs. Refer to the schematic inlet control nomograph shown in Figure 8-5.

1. Locate the selected culvert size (point 1) and flow rate (point 2) on the appropriate scales of the inlet control nomograph. For box culverts, the flow rate per foot of barrel width is used.
2. Using a straightedge, carefully extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (*HW/D*) scale (point 3). The first *HW/D* scale is also a turning line.
3. Each scale represents a different inlet type. If another *HW/D* scale is required, extend a horizontal line from the first *HW/D* scale to the desired scale and read the result.
4. Multiply *HW/D* by the culvert height, *D,* to obtain the required headwater depth, *HW*, from the invert of the control section to the energy grade line. If the approach velocity is neglected, *HW* equals the required headwater depth, HW_i. If the approach velocity is included in the calculations, deduct the approach velocity head from *HW* to determine HW_i.

5. Calculate the required depression (FALL) of the inlet control section below the stream bed using Equations 8.1 and 8.2:

$$HW_d = EL_{hd} - EL_{sf} \tag{8.1}$$

$$FALL = HW_i - HW_d \tag{8.2}$$

Where:

HW_d = design headwater depth (ft)
EL_{hd} = design headwater elevation (ft)
EL_{sf} = elevation of the stream bed at the face (ft)
HW_i = required headwater depth (ft)

Possible results and consequences of this calculation are:

a. If the FALL is negative or zero, set FALL equal to zero and proceed to step 6.
b. If the FALL is positive, the inlet control section invert must be depressed below the streambed at the face by that amount. If the FALL is acceptable, proceed to step 6.
c. If the FALL is positive and greater than is judged to be acceptable, select another culvert configuration and begin again at step 1.

6. Calculate the inlet control section invert elevation:

$$EL_i = EL_{sf} - FALL \tag{8.3}$$

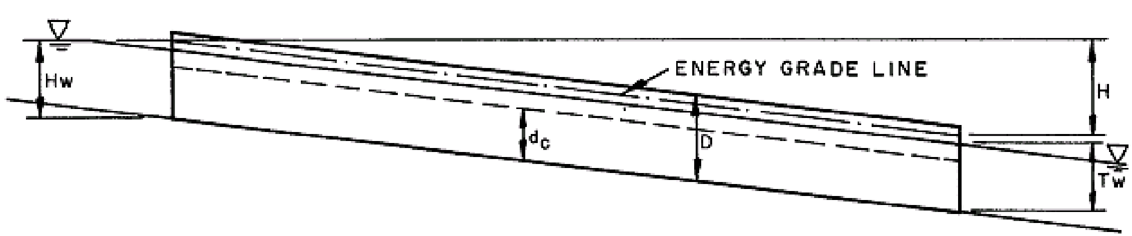
Where EL_i is the invert elevation at the face of a culvert (or throat for a tapered inlet).

8.2.3 Outlet Control

Outlet control flow occurs when the culvert barrel cannot convey as much flow as the inlet opening will accept. The control section for the outlet control flow in a culvert is located at the barrel outlet or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. All of the geometric and hydraulic characteristics of the culvert affect its capacity, including all factors governing inlet control, the water surface elevation at the outlet, and the slope, length, and hydraulic roughness of the culvert barrel.

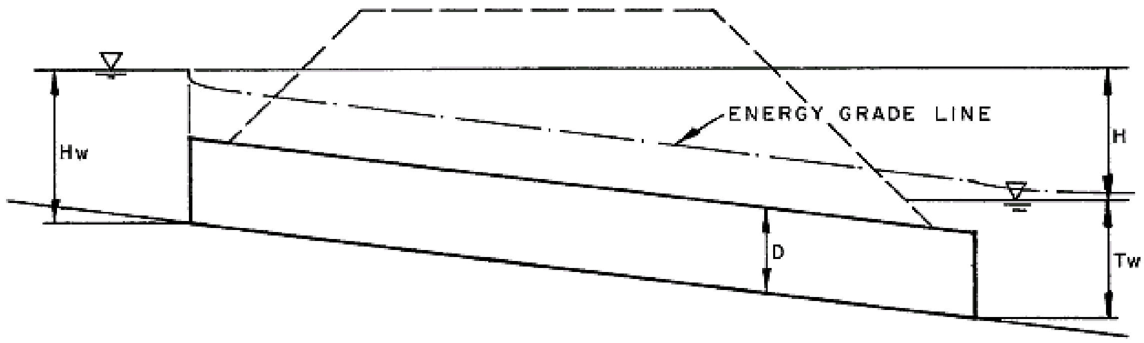
Outlet control will exist under two conditions. The first and least common is when the headwater is insufficient to submerge the top of the culvert, and the culvert slope is subcritical, shown in Figure 8-3.

Figure 8-3
Partially Full Conduit



The most common condition exists when the culvert is flowing full, shown in Figure 8-4.

Figure 8-4
Full Conduit



Under outlet control, culverts may flow full or partly full depending on various combinations of the above factors. Performance of a culvert under outlet control is affected by culvert length, roughness, and tailwater depth.

Outlet Control Calculation

Outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert under outlet control conditions. The approach and downstream velocities may be included in the design process. Entrance loss coefficients are listed in Table 8-2. Critical depth charts and outlet control nomographs used in the design process for typical configurations are included in Figures 8-11 through 8-16. For other configurations, refer to HDS-5 (FHWA 2005).

Use of the outlet control nomographs is described in the following paragraphs. Refer to the schematic critical depth chart and outlet control nomograph shown in Figures 8-9 and 8-10, respectively.

1. Determine the tailwater depth, TW, above the outlet invert at the design flow rate. This is obtained from backwater or normal depth calculations, or from field observations.
2. Enter the appropriate critical depth chart, Figure 8-9, with the flow rate and read the critical depth, d_c. The critical depth, d_c, cannot exceed the culvert diameter, D. The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for $d_c > 0.9D$ consult the *Handbook of Hydraulics* (Brater and King 1996) or other hydraulic reference.
3. Calculate $(d_c + D)/2$.
4. Determine the depth from the culvert outlet invert to the hydraulic grade line, h_o.

   $$h_o = TW \text{ or } (d_c + D)/2, \text{ whichever is larger} \quad (8.4)$$
5. From Table 8-2, obtain the appropriate entrance loss coefficient, K_e, for the culvert inlet configuration.
6. Determine the head losses through the culvert barrel, *H*, using the outlet control nomograph, Figure 8-10.

a. Required Manning's *n* values are presented in Table 5-1 of Section 5, Storm Drains. If the Manning's *n* value given in the outlet control nomograph is different than the required Manning's *n* for the culvert, adjust the culvert length using Equation 8.5:

$$L_1 = L\,(n_1/n)^2 \qquad (8.5)$$

Where:

L_1 = adjusted culvert length (ft)
L = actual culvert length (ft)
n_1 = desired Manning's n value
n = Manning's *n* value from the outlet control chart

b. Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate k_e scale (point 2). This defines a point on the turning line (point 3).

c. Again using the straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the head loss, *H*, scale. *H* is the energy loss through the culvert, including entrance, friction, and outlet losses. Careful alignment of the straightedge is necessary to obtain good results from the outlet control nomograph.

7. Calculate the required outlet control headwater elevation, EL_{ho}.

   $EL_{ho} = EL_o + H + h_o$, where EL_o is the invert elevation at the outlet
   (8.6)

8. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

8.2.4 Computer Applications

Although the nomographs discussed in this section are still used, increasingly culverts are designed using computer applications. Among these applications are the FHWA's HY8 Culvert Analysis, Bentley's CulvertMaster, and numerous other proprietary applications. If a computer application other than HY8 or CulvertMaster is to be used, the designer must first get approval from the City. The designer must show that these criteria have been followed regardless of the computer or other model chosen.

Table 8-2
Entrance Loss Coefficient, K_e

Type of Structure and Design of Entrance	Coefficient, K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° - 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwalls at 10° - 25° to barrel, square-edged at crown	0.5
Wingwalls parallel, square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*"End Section conforming to fill slope" are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

8.3 CULVERT MATERIALS, INLET AND OUTLET CONFIGURATION

Culvert analysis and design shall consider design flow, culvert size and material, upstream channel and entrance configuration, downstream channel and outlet configuration, and erosion protection.

8.3.1 Construction Material and Pipe Size

Culverts may be constructed of high-density polyethylene (HDPE) pipe, pre-cast concrete, or cast-in-place concrete. Culverts shall be designed to carry storm flows without accumulating sediment.

The minimum pipe size for culverts under streets shall be an 18-inch diameter round pipe or shall have an equivalent 18-inch diameter round cross-sectional area for other shapes. Driveway culverts shall be sized to pass the Minor storm peak flow for the adjoining street classification without overtopping the driveway and shall have a minimum diameter of 15 inches. Driveway culverts shall be designed for driveway traffic loads and shall be placed with adequate cover to protect the culvert. The minimum depth of bury shall be 9 inches.

8.3.2 Location

Culverts shall be located on existing stream thalwegs and aligned to give the stream a direct entrance and direct exit. Abrupt changes in direction at either end may retard the flow and make a larger structure necessary. If this is not practical and the water must be channeled into the culvert, headwalls, wingwalls, and aprons shall be used as protection against scour and to provide an efficient inlet.

Where the natural alignment would result in an exceptionally long culvert, modification to the natural alignment may be necessary. Since such modifications will change the natural stability of the channel, an investigation into other options is recommended.

Roadway alignment also affects culvert design. The vertical height of the roadway may restrict the maximum culvert diameter that can be used. Low vertical clearance may require the use of elliptical or arched culverts, or the use of a multiple-barrel culvert system. All culverts shall have a minimum of 1.5 feet of cover from top of surface course to outside top of pipe, with no part of the pipe in the pavement section. Culverts for which less than 1.5 feet of cover is available will require independent structural analysis and other provisions (i.e., full depth concrete fill around pipe to compensate for the loss of proper cover).

8.3.3 Inlet Configuration

One of the most important considerations in the design of a culvert is the inlet configuration. Since the natural channel is usually wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will lessen the energy loss and thus create a more hydraulically efficient inlet condition. The inlet type can also increase the overall structural integrity by retaining fill slope and preventing inlet scour with subsequent undermining of the culvert. All culverts 54 inches in diameter or larger shall be designed with headwalls and wingwalls. Headwalls and wingwalls or flared-end sections may be used at the inlet and outlet of culverts 48 inches in diameter and smaller.

Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. The primary advantage of projecting inlets is relatively low cost. Bell-and-spigot concrete pipe or tongue-and-groove concrete pipe with the bell end or grooved end used as the inlet section are quite efficient hydraulically, having an entrance coefficient of approximately 0.2, Table 8-2. For concrete pipe that has been cut, the entrance is square-edged, and the entrance coefficient is approximately 0.5, Table 8-2.

Headwalls

Headwalls may be used for increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used. For tongue-and-groove or bell-end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reasons for using headwalls are embankment protection and ease of maintenance.

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and where the culvert is skewed to the normal channel flow. Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use shall be justified for reasons other than an increase in hydraulic efficiency.

Headwalls and wingwalls shall be designed by a registered structural engineer and structural design calculations submitted for review and approval.

Tapered Inlets

Inlet configuration is a primary factor influencing the performance of a culvert operating under inlet control. Inlet edges can cause severe contraction of the flow, as in the case of a thin edge, projecting inlet. In a flow contraction, the effective cross-sectional area of the barrel may be reduced to about one half of the actual available barrel area. As the inlet configuration is improved, the flow contraction is reduced, thus improving the performance of the culvert.

A tapered inlet is flared, with an enlarged face section and a hydraulically efficient throat section. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because the simple beveled edge is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet. HDS-5 (FHWA 2005) provides guidance on the design of improved inlets.

8.3.4 Trash Racks

The use of trash racks or gratings at inlets to culverts and long underground pipes shall be considered on an individual basis. While trash racks and gratings are used for safety reasons, experience has clearly shown that when the culvert is operating during heavy runoff, normal gratings often become clogged and the culvert is rendered ineffective. Engineering judgment shall be used to determine whether trash/safety racks are necessary, and concurrence for their use must be provided by the City Engineer. Factors influencing use of trash racks include:

1. Tributary land use (urban, rural, forest)
2. Location (urban/rural)
3. Design flow
4. Size of culvert
5. Anticipated debris loading
6. Performance of nearby existing structures
7. Public safety

Generally, trash racks are needed on culverts if one cannot see clearly through the culvert, if the culvert is more than 150 feet long, if a 48 inch diameter object cannot pass through, or if the outlet may trap or injure a person being carried through.

When used, the open area through the grate at the design water surface shall be a minimum of 4 times the design flow area of the culvert. Trash racks shall be constructed of smooth steel pipe, with a corrosion protection finish and capable of withstanding the full hydraulic load when completely blocked under maximum submergence. Bar clear spacing shall not exceed 6 inches, and bars shall be generally perpendicular to flow. The longitudinal slope of the trash rack shall not exceed 3(H):1(V). There shall be a minimum clear area under the front edge of 9 to 12 inches. Trash racks shall be attached by removable devices such as bolts or hinges to allow access for maintenance, prevent undesirable access, and prevent vandalism. Trash racks shall be installed only at inlets to drainage conduits. Trash racks at outlets to drainage conduits can pose a safety issues and trap debris, reducing capacity and causing maintenance problems. The rack must not cause water to rise higher than the maximum allowable flood elevation. A calculation may be needed to estimate the backwater due to the trash rack that is separate from the backwater due to the culvert. The total straining area of a rack should be at least ten times the cross-sectional area of the culvert being protected. HEC-9 (FHWA 2005) should be consulted on the analysis and design of the trash rack.

8.3.5 Outlet Configuration

Scour at culvert outlets is a common occurrence because culverts confine the natural channel flow and increase the velocity resulting in potentially erosive capabilities at the outlet of the barrel. Turbulence and erosive eddies form as the flow expands back into the natural channel. The characteristics of the channel bed and bank material, the tailwater conditions at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Methods for predicting scour hole dimensions at culvert outlets can be found in Chapter 5 of HEC No. 14, "Hydraulic Design of Energy Dissipaters for Culverts and Channels" (FHWA 1996).

Culvert outlets must be designed to safely transition storm flows back to the downstream channel and mitigate conditions which can produce scour. When detrimental scour is expected, protective measures must be designed. Protection against scour at culvert outlets varies from limited riprap placement to complex and expensive energy dissipation devices. Other techniques to reduce scour potential include use of a rougher culvert material, internal energy dissipation rings, or designing the culvert on a flatter slope. Providing preformed scour holes, which approximate the configuration of naturally formed scour holes, dissipate energy while providing a protective lining to the stream bed. Concrete aprons used with riprap lined channel expansion sections protect the channel and redistribute and spread the flow (See Section 4.4.9). Headwalls and cutoff walls can be used to protect the integrity of the culvert and the surrounding fill. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices, which include hydraulic jump basins, impact basins, drop structures, and stilling wells. Design information for the general types of energy dissipators is provided in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 1983 and 2000).

8.3.6 Easements

Culverts shall be designed such that all appurtenances, including headwalls, wingwalls, and erosion protection is within public ROW. Culverts extending beyond available public ROW shall be within easements with a width of twice the depth to the culvert invert at the right-of-way line (at the deepest point) or 20 feet wider than the culvert width, whichever is greater. The culvert easement shall extend 10 feet beyond any end treatments or protection materials or 20 feet beyond the end of the culvert, whichever is greater.

8.4 DESIGN EXAMPLE

Problem: Determine the culvert size necessary to convey the 100-year discharge in Doe Creek beneath John Boulevard. The results of the analysis are shown in the Culvert Design Form of Figure 8-17.

- Top of road elevation = 4928 ft
- Culvert inlet invert elevation = 4920 ft
- Culvert outlet invert elevation = 4918 ft
- Culvert length = 200 ft
- Inlet = Groove end with headwall and wingwalls at 45°
- Outlet = Groove end with headwall and wingwalls at 45°
- Flow = 191 cfs
- Tailwater Depth = 4 ft

Solution:

Step 1: Assume a reinforced concrete pipe diameter and determine the headwater-to-depth ratio for inlet control from Figure 8-6. Assume a reinforced concrete pipe diameter of 5 feet. Given are a 100-year design flow of 191 cfs and a grooved end with headwall inlet configuration.

HW/D = 1.28 see Figure 8-18

Step 2: Calculate the required headwater depth assuming inlet control conditions. Multiply the pipe diameter times the headwater to depth ratio.

Headwater = HW_i = D*(HW/D) = 5.0*1.28 = 6.4 ft

Step 3: Calculate the required headwater elevation assuming inlet control conditions.

FALL = 0, since the culvert invert is at grade

$HW_i = HW_d$

$EL_i = EL_{sf}$

$EL_{hd} = EL_i + HW_i = 4920 + 6.4 = 4926.4$ ft

Step 4: Estimate the critical depth, d_c, in the culvert from Figure 8-11.

d_c = 3.9 ft (see Figure 8-19)

Step 5: Since the tailwater depth is less than the culvert diameter, compute the estimated water depth at the culvert outlet assuming the tailwater does not control the outlet conditions.

Outlet Depth = $(d_c + D)/2 = (3.9+5.0)/2 = 4.5$ ft

Step 6: Determine the flow depth at the culvert outlet, h_o. The estimated depth is the maximum value of the tailwater depth and the water depth assuming no tailwater.

4.5 ft > 4.0 ft, therefore $h_o = 4.5$ ft

Step 7: Estimate the head, H, for outlet control conditions from Figure 8-14.

H = 2.6 ft (see Figure 8-20)

Step 8: Calculate the required outlet control headwater elevation, EL_{ho}.

$EL_{ho} = EL_o + H + h_o = 4918 + 2.6 + 4.5 = 4925.1$ ft

Step 9: Determine if the culvert is under inlet control or outlet control and provide the resulting control headwater elevation and depth.

$EL_{hd} > EL_{ho}$ (4926.4 > 4925.1), the culvert is under inlet control

Control Headwater Elevation = 4926.4

HW = 6.4 ft

Step 10: Calculate the outlet velocity by an appropriate method and determine the amount of outlet protection needed, see Section 8.2.1.

v = 10.0 ft/s

8.5 CHECKLIST

All of the design criteria in this chapter must be followed. Several key considerations include:

1. No street overtopping for the Minor storm.
2. Check minimum and maximum outlet velocity.
3. Minimum culvert size crossing the public ROW is 18-inch diameter or equivalent.
4. Minimum culvert size for roadside ditches at driveways is 15-inch diameter or equivalent.
5. Headwalls and wingwalls are provided for all culverts with diameter larger than 48 inches.
6. Check maximum headwater for design conditions.
7. Check structural requirements, buoyancy, and emergency overflow route.
8. Check public safety provisions.

Figure 8-5
Inlet Control Nomograph (Schematic)

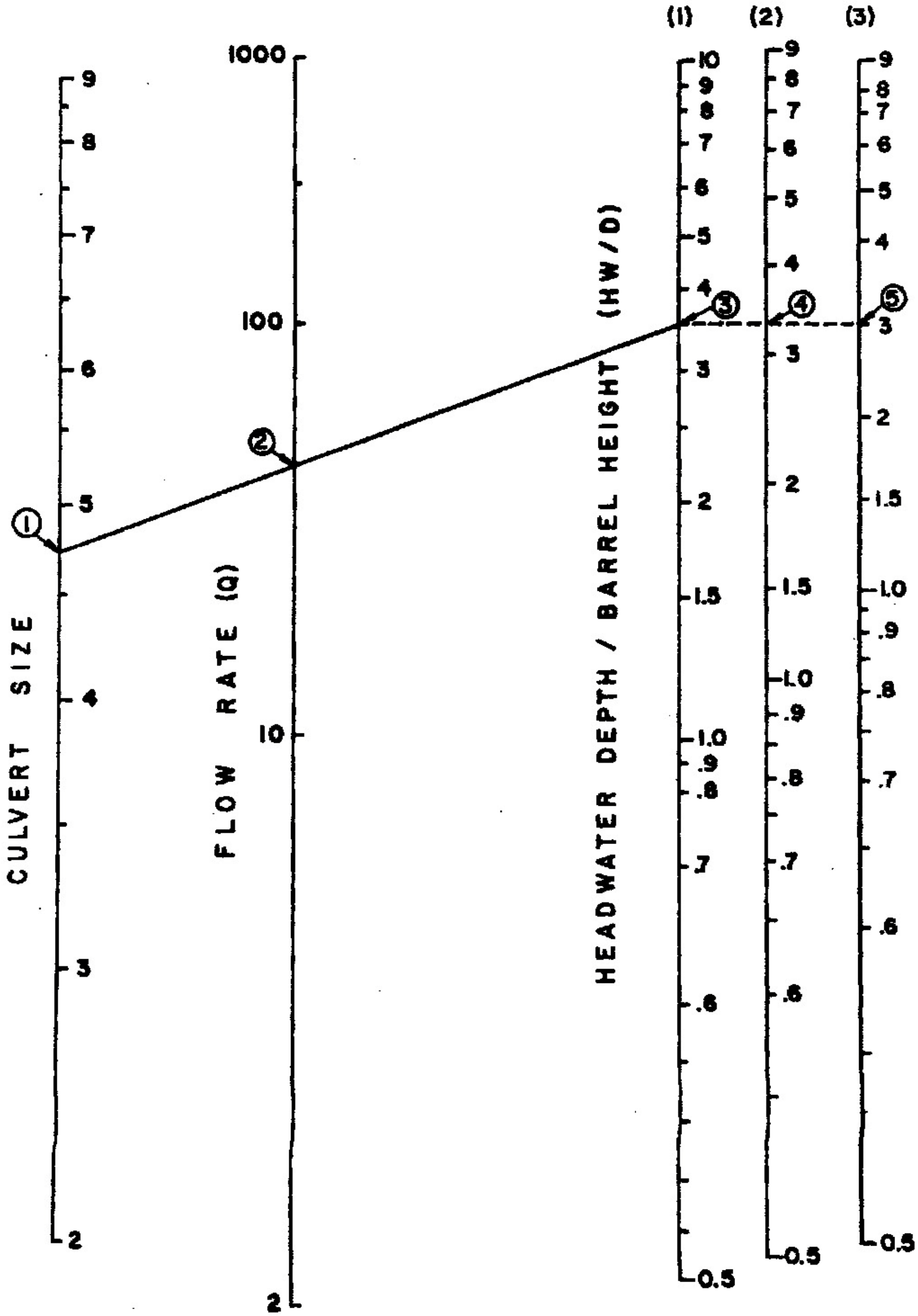


Figure 8-6
Headwater Depth for Concrete Pipe Culverts with Inlet Control

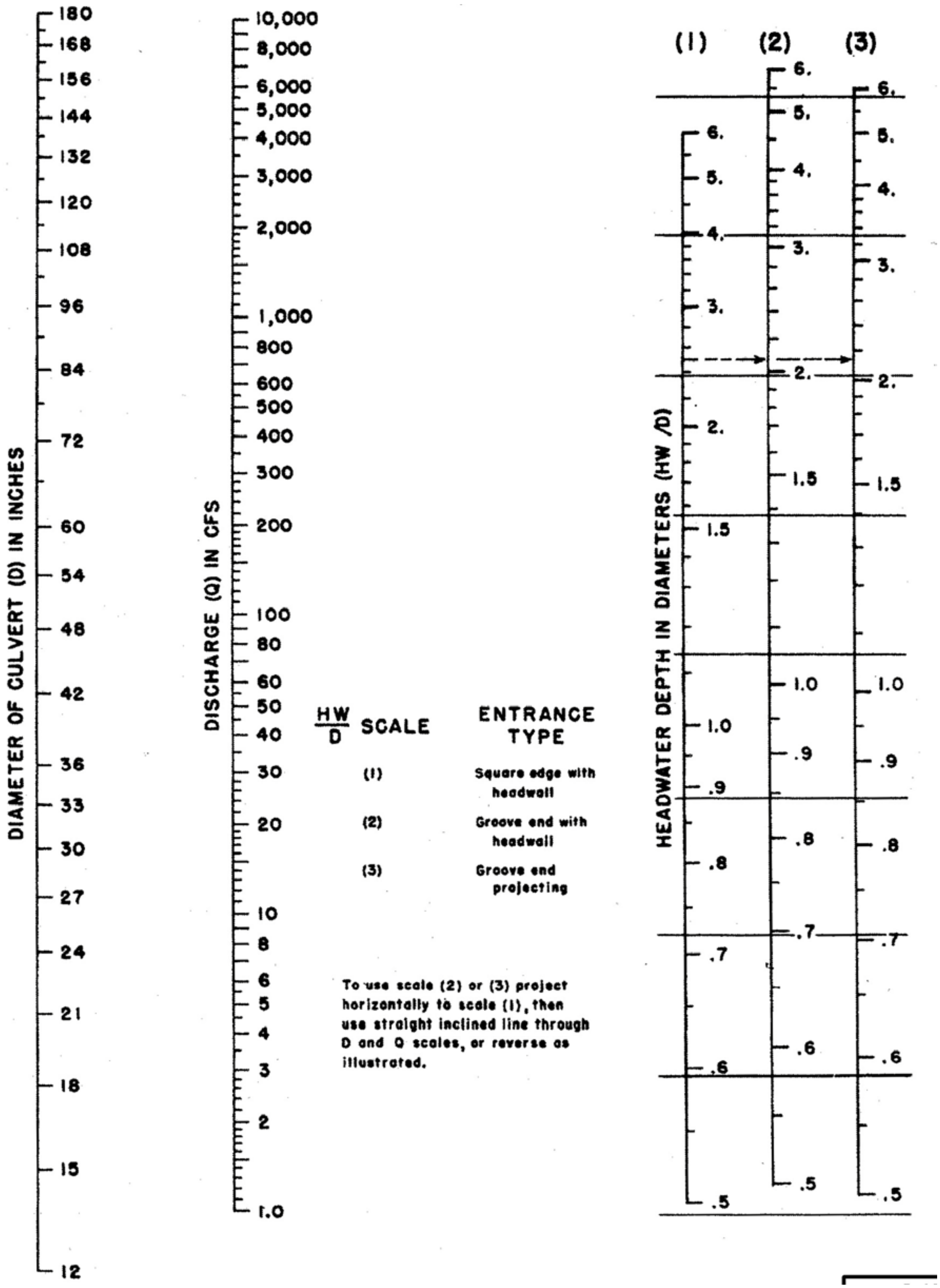


Figure 8-7
Headwater Depth for Concrete Box Culverts with Inlet Control

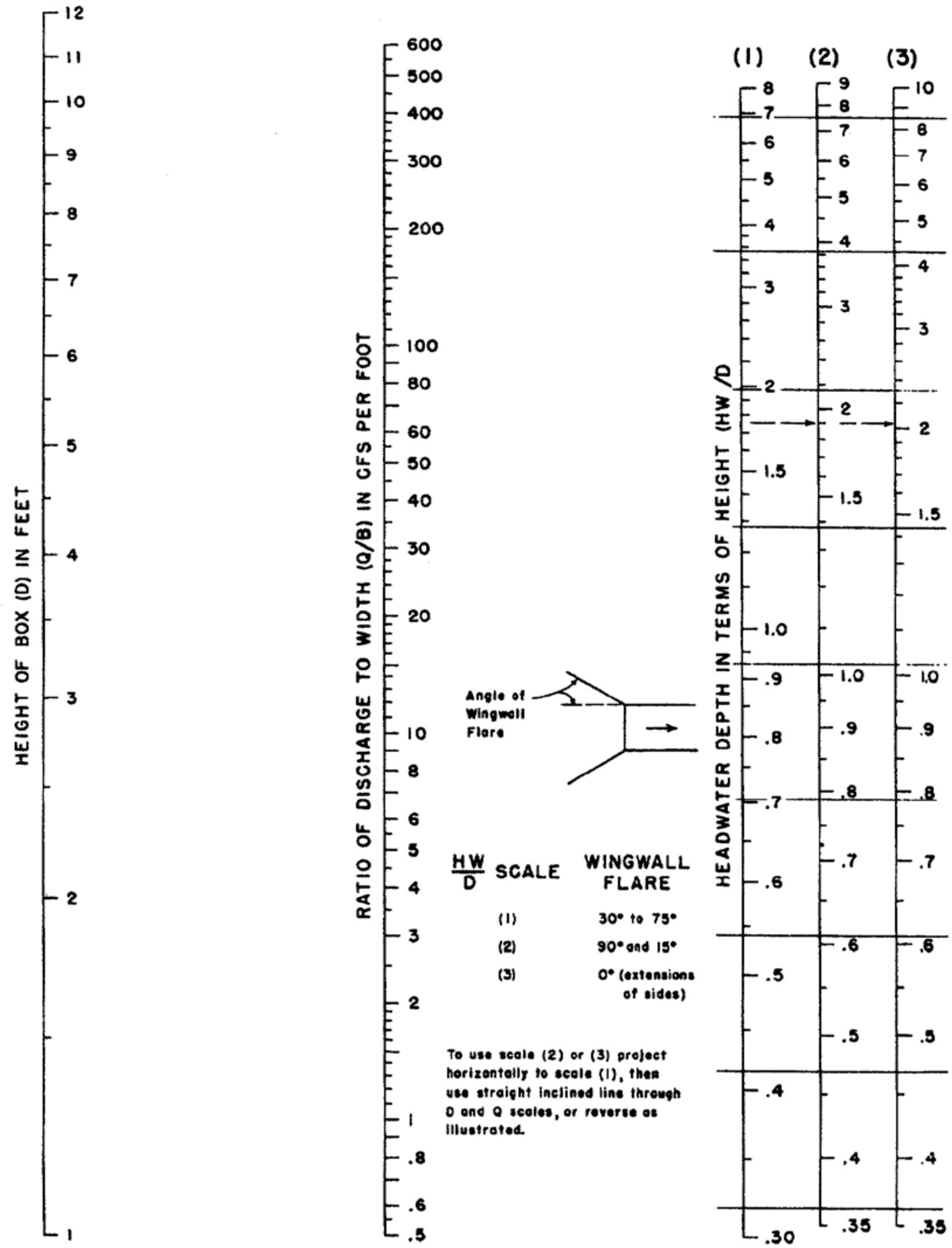


Figure 8-8
Headwater Depth for Oval Concrete Pipe Culverts with Inlet Control
(Long-Axis Horizontal)

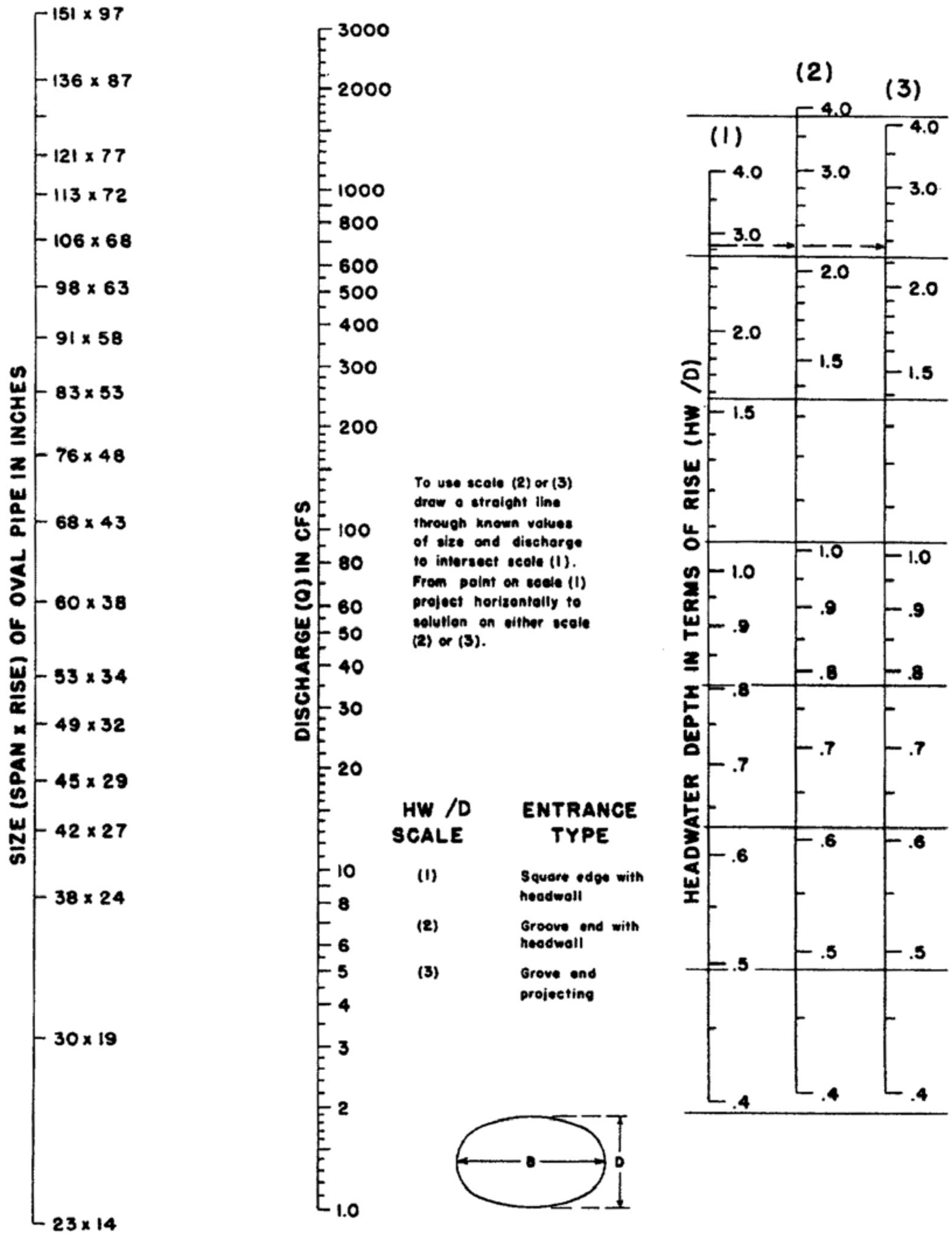


Figure 8-9
Critical Depth Chart (Schematic)

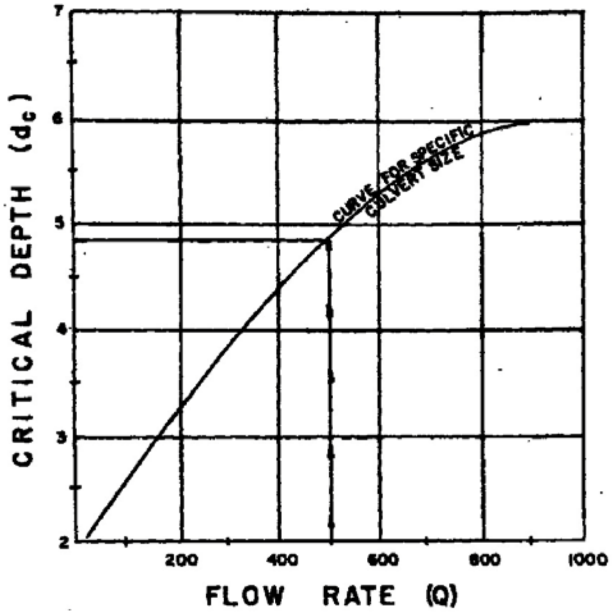


Figure 8-10
Outlet Control Nomograph (Schematic)

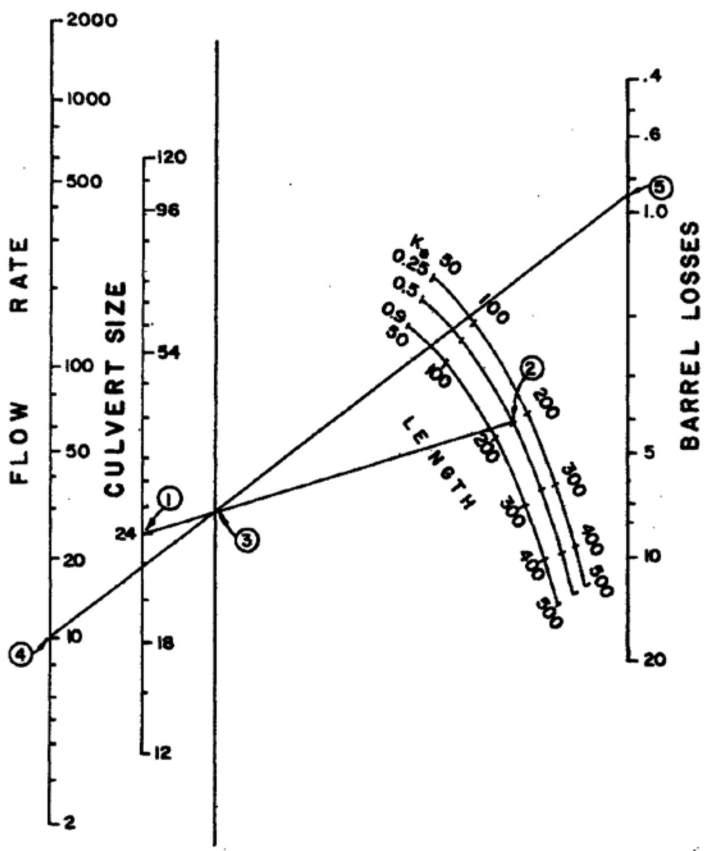


Figure 8-11
Critical Depth – Circular Pipe

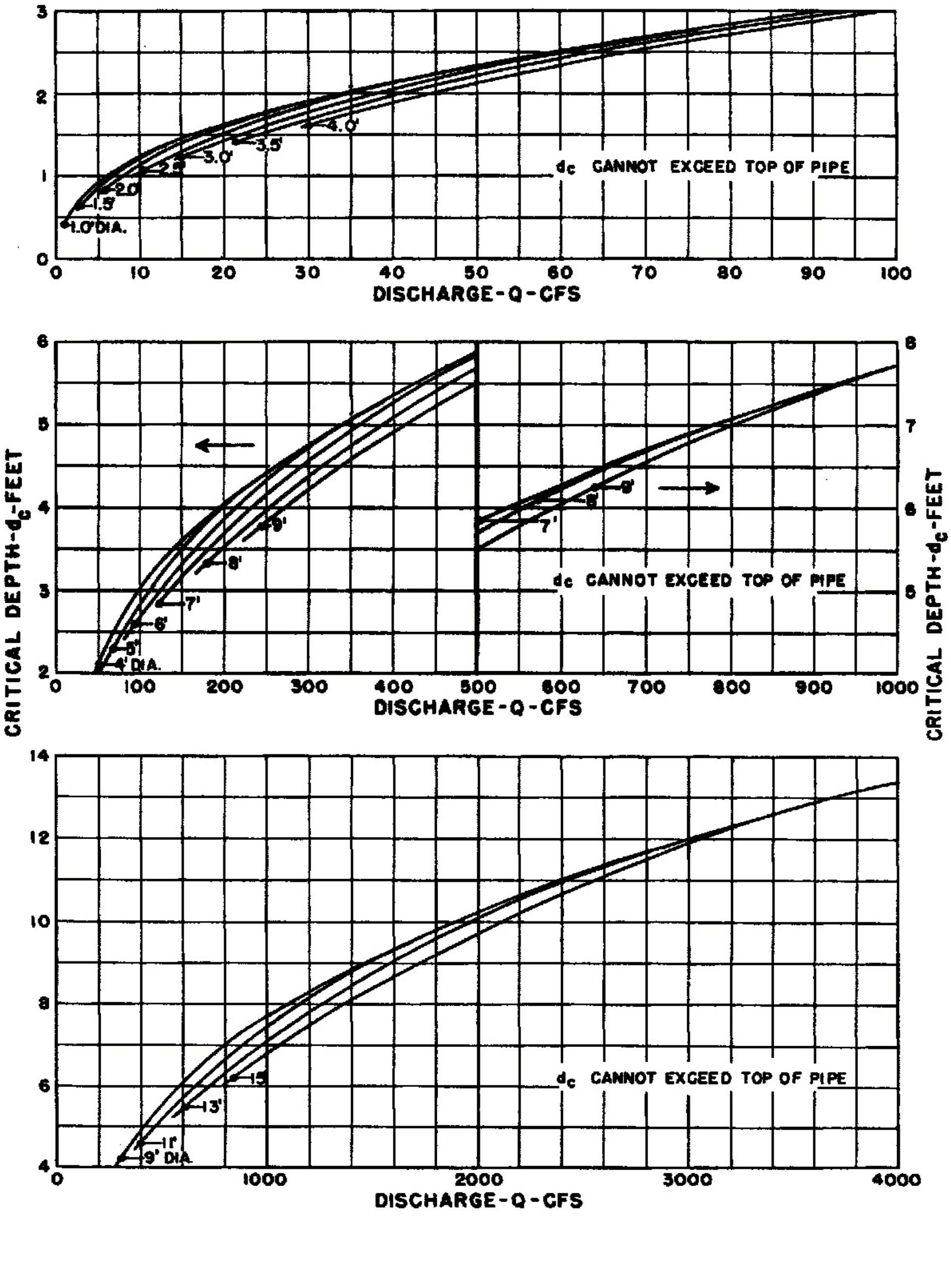


Figure 8-12
Critical Depth – Rectangular Section

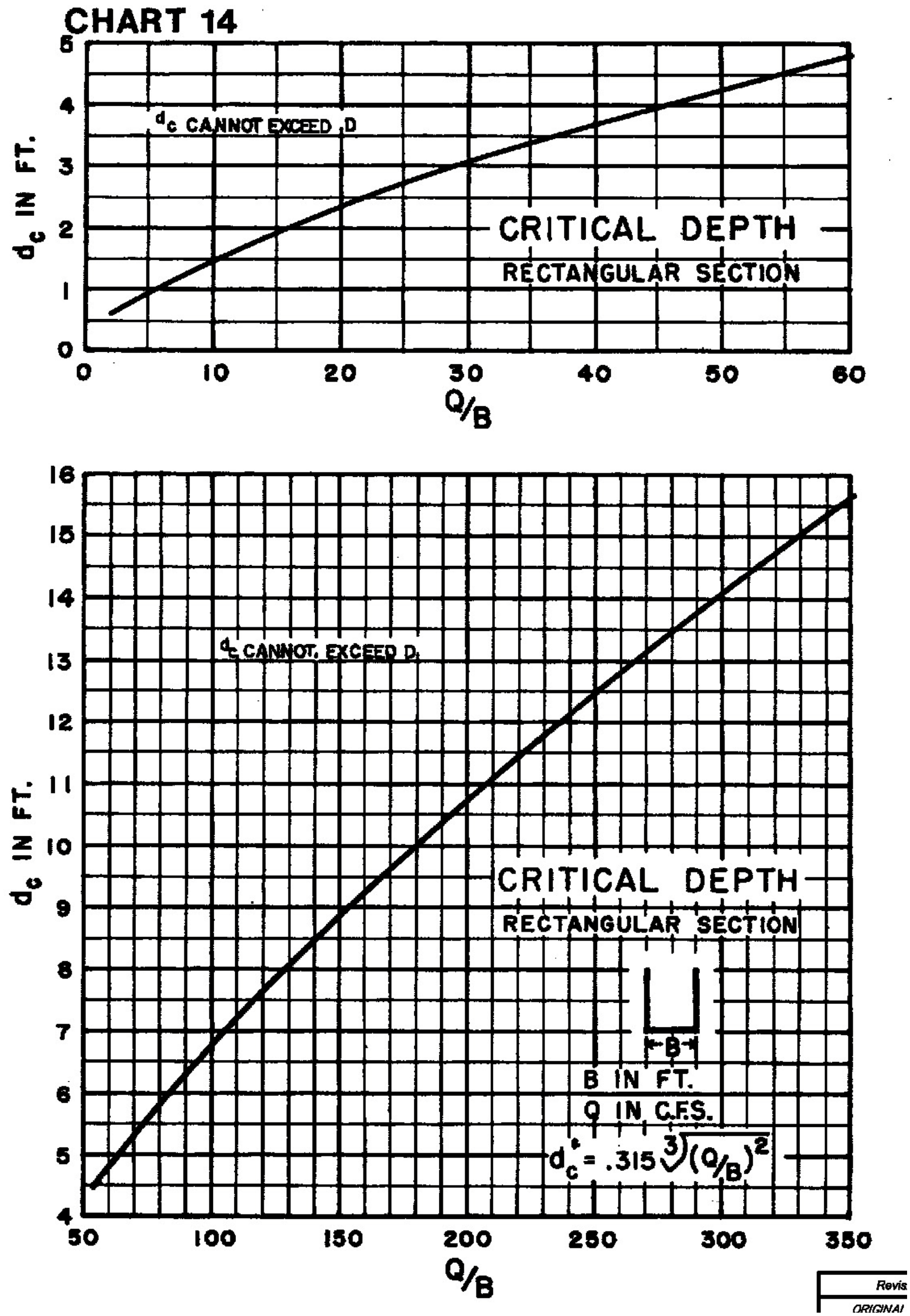


Figure 8-13
Critical Depth – Oval Concrete Pipe (Long-Axis Horizontal)

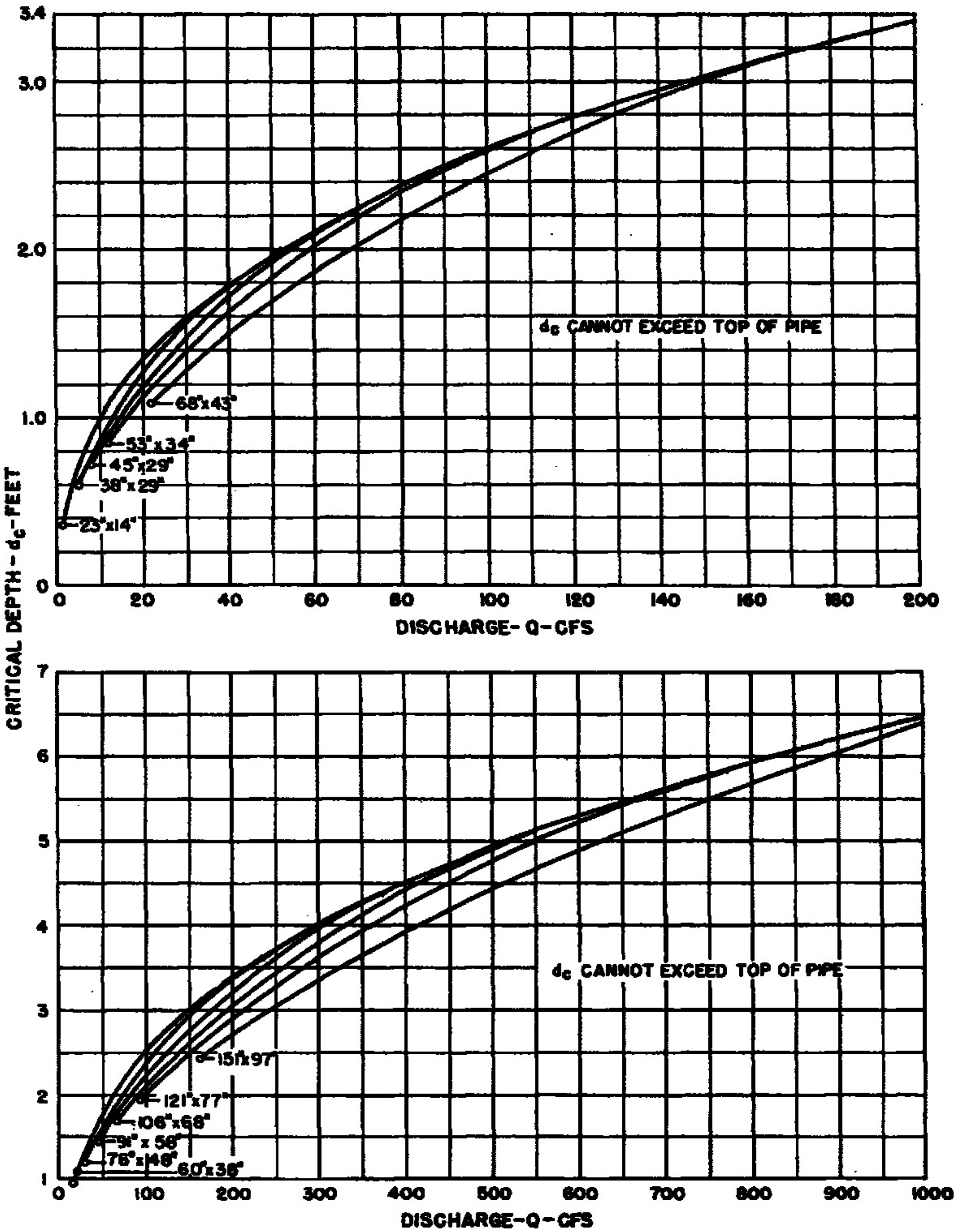


Figure 8-14
Head for Concrete Pipe Culverts with Outlet Control (n = 0.012)

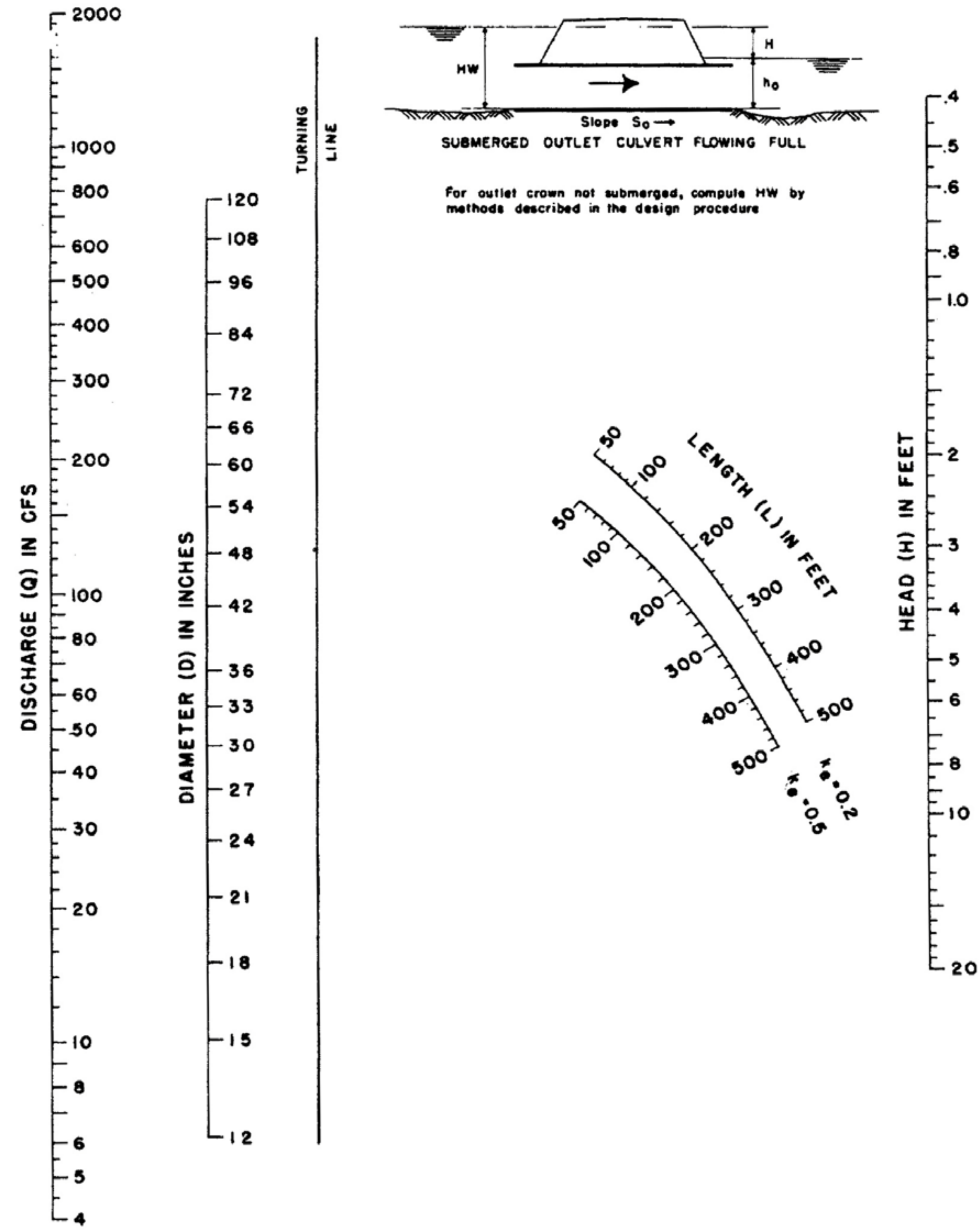


Figure 8-15
Head for Concrete Box Culverts with Outlet Control (n = 0.012)

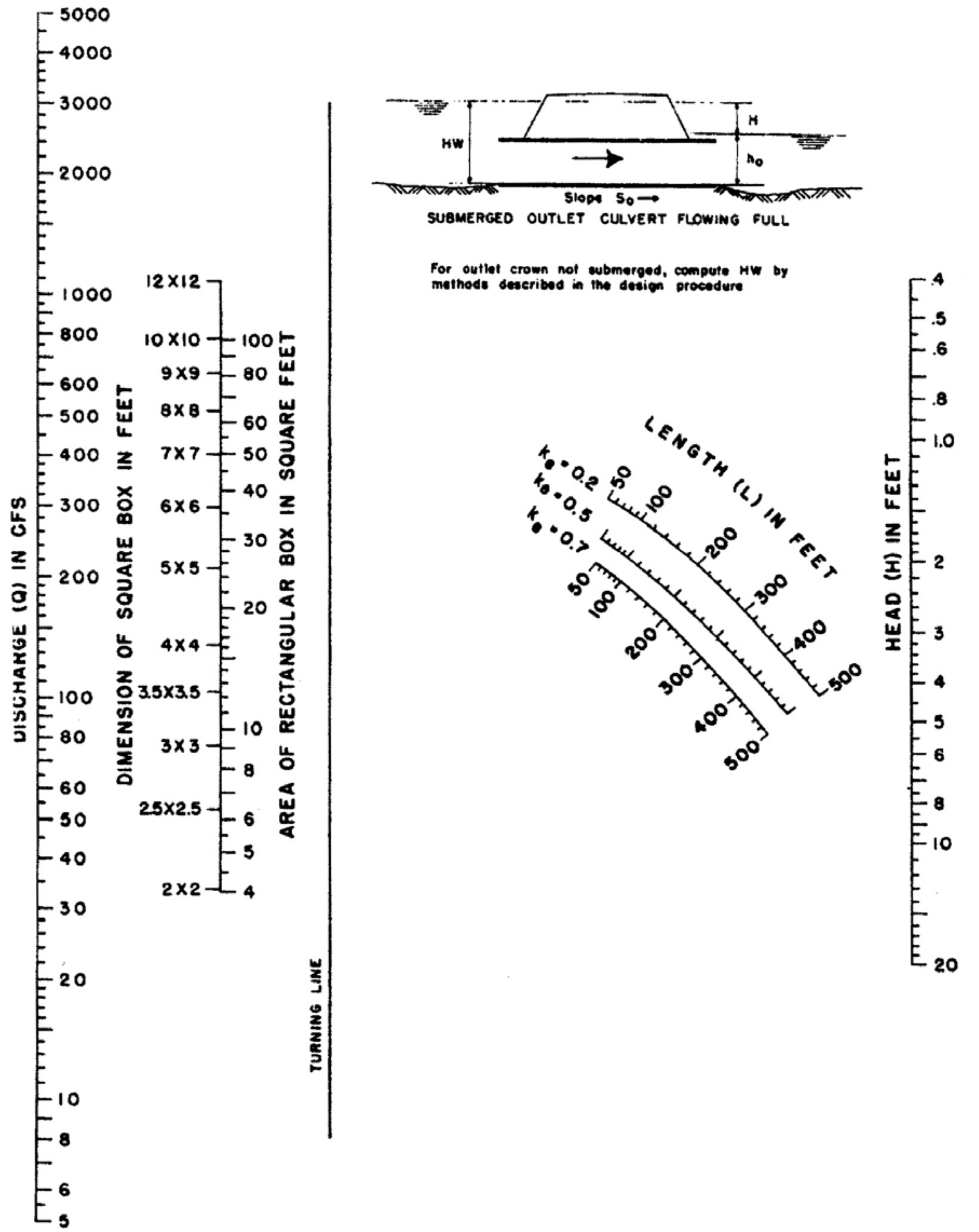


Figure 8-16
Head for Oval Concrete Pipe Culverts with Outlet Control, n = 0.012
(Long-Axis Horizontal or Vertical)

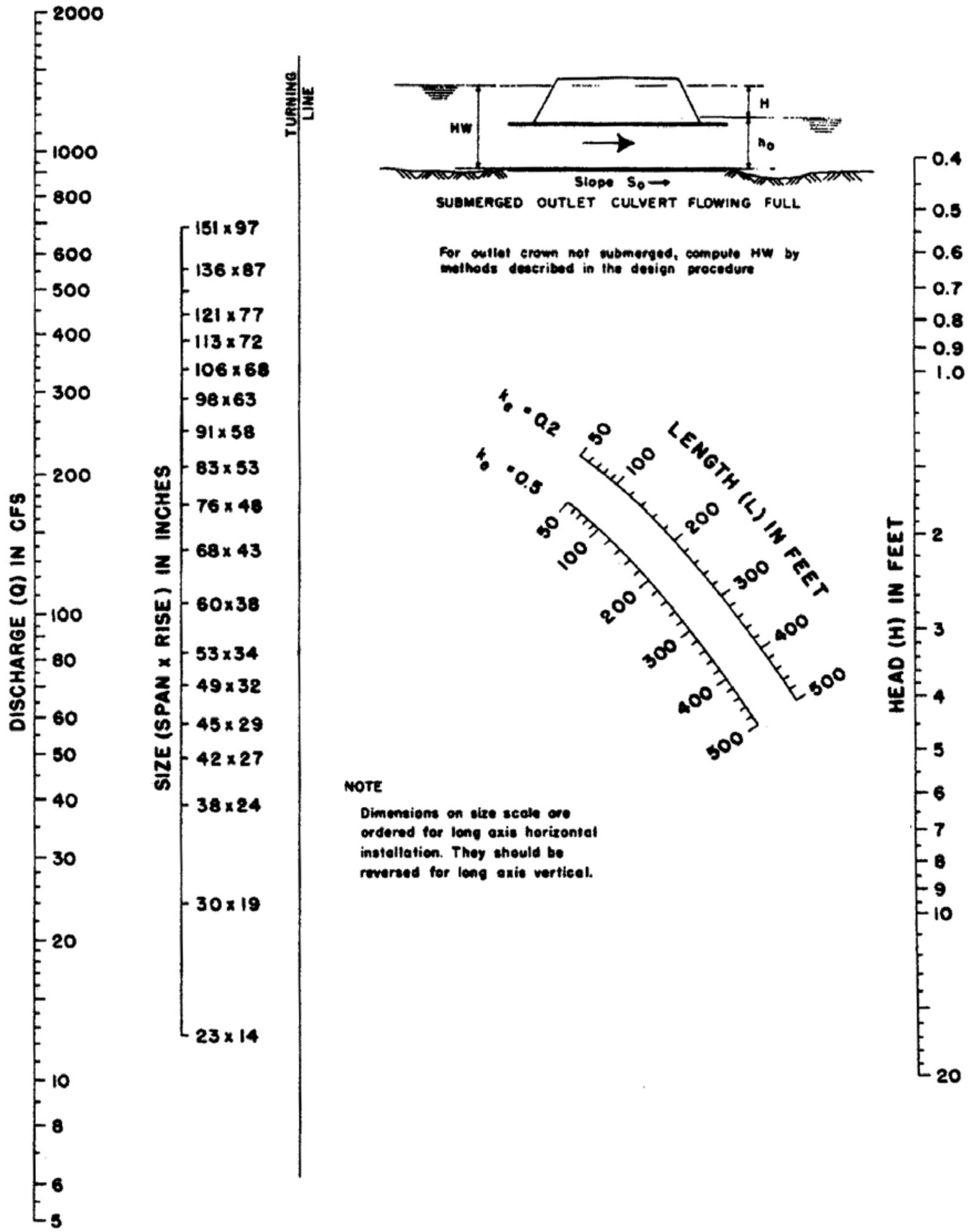
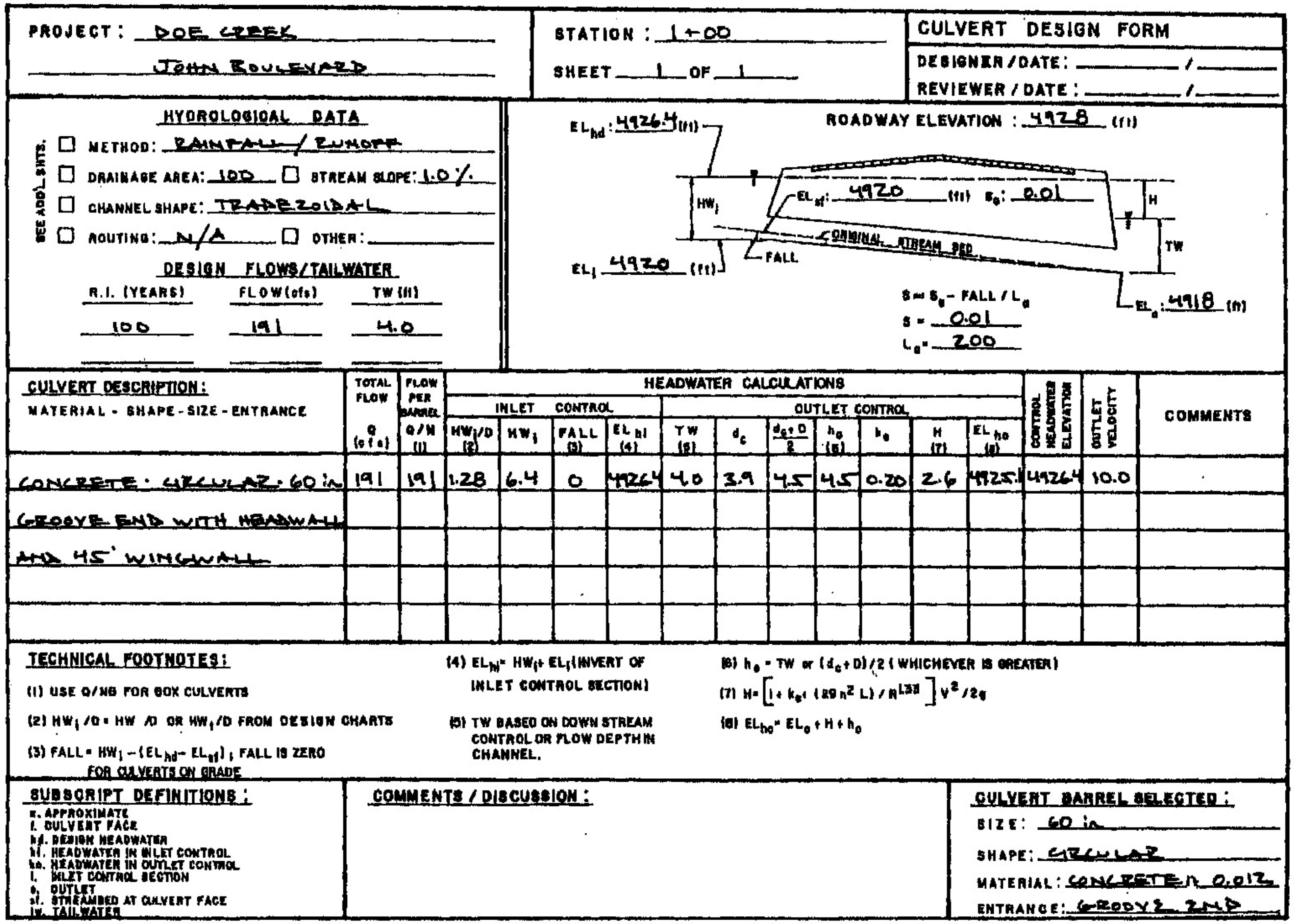


Figure 8-17
Example Culvert Design Form



CULVERT DESIGN FORM

PROJECT: DOE CREEK
JOHN BOULEVARD

STATION: 1+00
SHEET 1 OF 1

DESIGNER/DATE: ____ / ____
REVIEWER/DATE: ____ / ____

HYDROLOGICAL DATA (SEE ADD'L SHTS.)

- ☐ METHOD: RAINFALL / RUNOFF
- ☐ DRAINAGE AREA: 100 ☐ STREAM SLOPE: 1.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	191	4.0

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
CONCRETE · CIRCULAR · 60 in	191	191	1.28	6.4	0	4926.4	4.0	3.9	4.5	4.5	0.20	2.6	4925.1	4926.4	10.0	
GROOVE END WITH HEADWALL																
AND 45° WINGWALL																

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

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

SUBSCRIPT DEFINITIONS:

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

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: 60 in
SHAPE: CIRCULAR
MATERIAL: CONCRETE n 0.012
ENTRANCE: GROOVE END

Figure 8-18
Example Inlet Control Nomograph

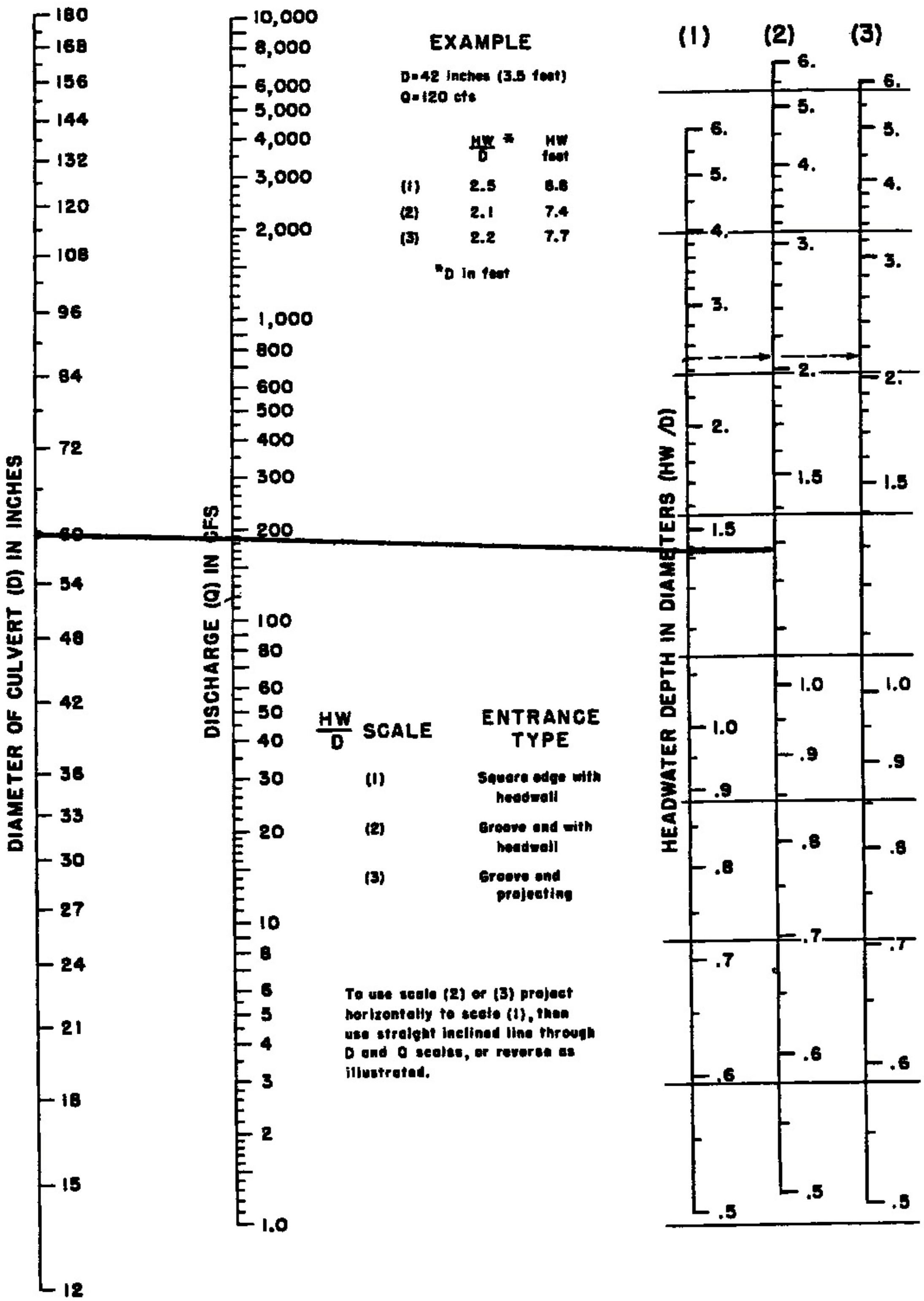


Figure 8-19
Example Critical Depth

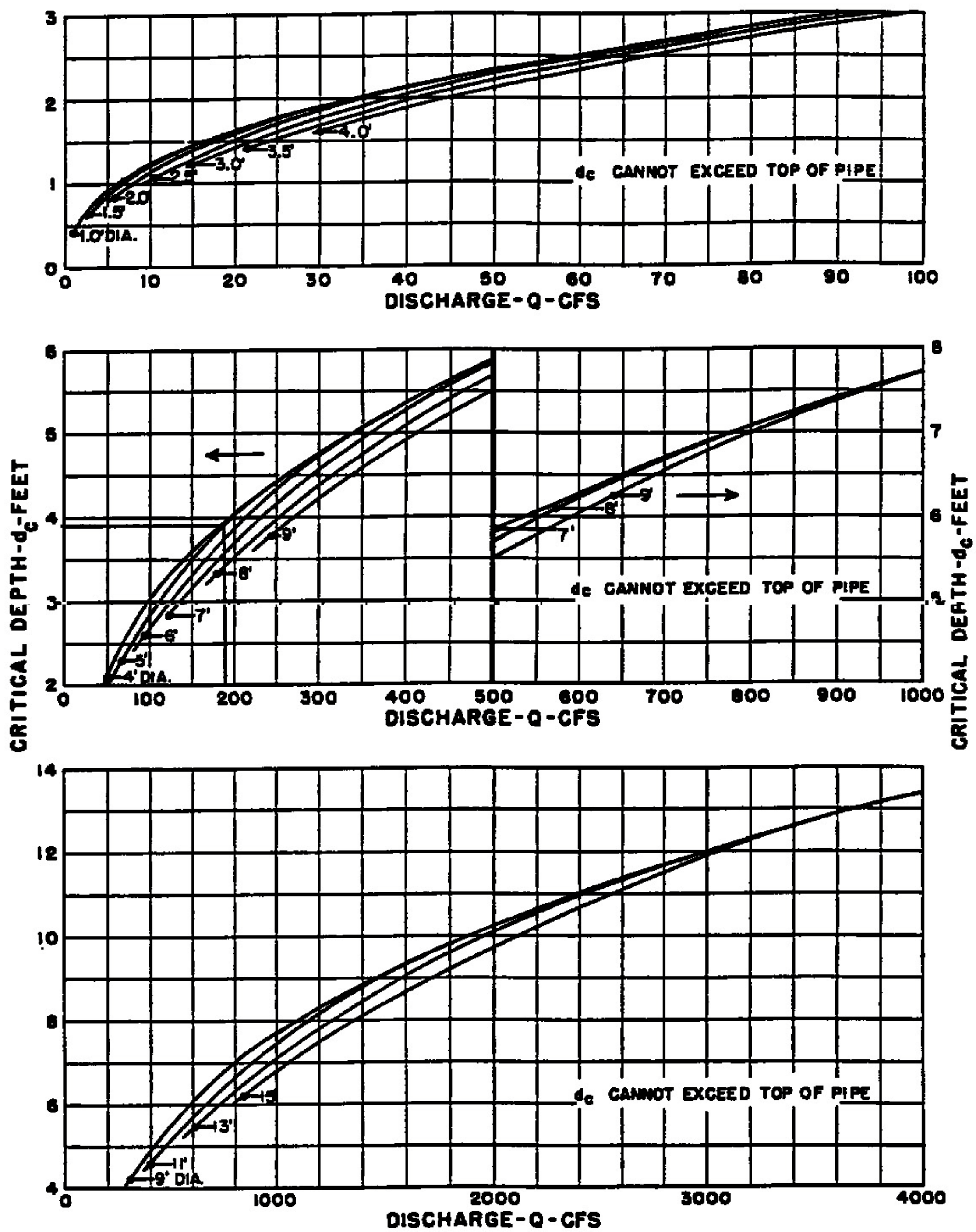
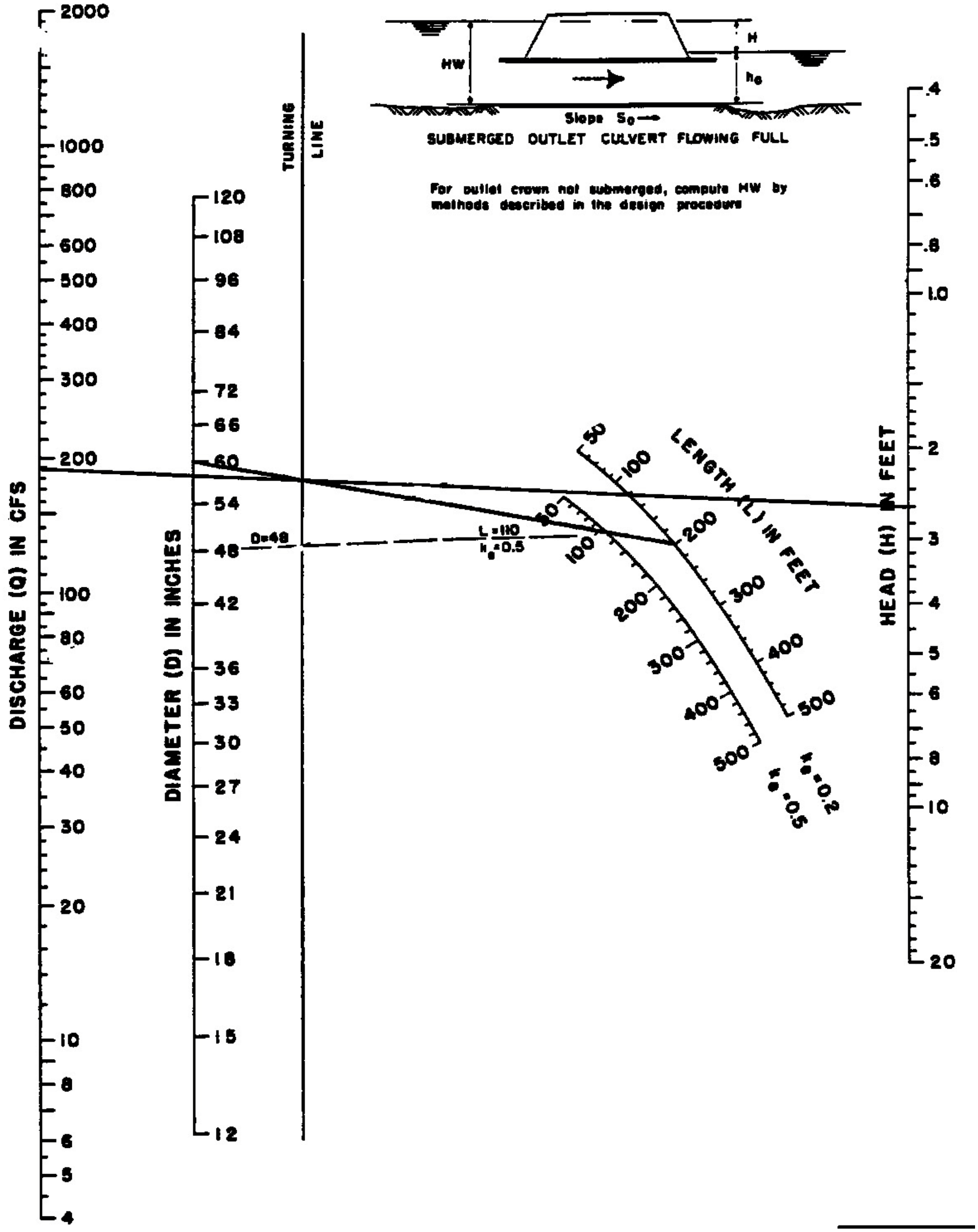


Figure 8-20
Example Outlet Control Nomograph



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SECTION NINE
HYDRAULIC STRUCTURES

9.1 INTRODUCTION

Hydraulic structures (e.g., riprap, channel grade control structures, bridges) help to control the energy associated with flowing water, thereby reducing erosion-related damage to the drainageway. The proper application and design of hydraulic structures can reduce initial and future maintenance costs by managing the characteristics of the flow to fit the project needs and environmental constraints. All hydraulic structures should be designed and constructed considering aesthetics and should fit in with their surroundings to the greatest extent practicable. Structures must be designed with long-term maintainability as a key criterion, and consideration must be given to environmental, ecological, aesthetic and public safety objectives. These factors should be integrated with careful and thorough hydraulic engineering design to produce a cost effective structure.

The design of hydraulic structures shall be in accordance with the criteria presented in Table 9-1 and in the following paragraphs.

Table 9-1
Criteria for Hydraulic Structures

Hydraulic Structure	Application	Criteria
Riprap	Riprap can be placed at culverts, storm drain outlets, channel bottom and banks, check drops, bridges, and areas subject to erosion.	Chapter 4.4.9 of this Manual.
Channel Grade Control Structures (Check and Drop Structures)	Grade control structures, such as check structures for low flow channels and drop structures across waterways, dissipate energy and can effectively reduce upstream channel erosion and instability. Grade control structures are also often necessary to meet the maximum permissible velocity for major design storm runoff in grass-lined channels and wetland channels.	Section 9.3 of this Manual. "Open Channel Design Criteria" section of MAJOR DRAINAGE in UDFCD MANUAL.
Conduit Outlet Structures	Conduit outlet structures are designed to dissipate flow energy and reduce erosion at culverts and storm drain outlets. If an impact type energy dissipator is used on a culvert, an upstream trash rack must be provided.	FHWA's Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC-14).
Bridges	Bridge structures can cause adverse hydraulic effects and scour that must be evaluated and controlled in the hydraulic design.	Section 9.5 of this Manual and WYDOT Road Design Manual.
Transitions and Constructions	Channel transitions are typically used to alter the cross-sectional geometry for specific purposes such as fitting the waterway within a more confined right-of-way. Constrictions such as bridges and culverts must be planned to meet hydraulic design goals.	"Transitions and Constrictions" section of HYDRAULIC STRUCTURES chapter in USDCM
Rundowns	A rundown is used to convey storm runoff from high on the bank of an open channel to the low-flow channel of the drainageway or into a detention facility. The purpose is to control erosion and head cutting from concentrated flow. Without such rundowns, the concentrated flow will create erosion.	"Rundown" section of HYDRAULIC STRUCTURES in USDCM

9.2 RIPRAP

Riprap can be placed at culverts, storm drain outlets, channel bottom and banks, check drops, bridges, or other areas subject to erosion. Criteria for design of riprap revetment are in Section 4.4.9 Riprap Lined Channels.

9.3 CHANNEL GRADE CONTROL STRUCTURES

Grade control structures, such as check structures for low flow channels and drop structures across waterways, dissipate energy and can effectively reduce upstream channel erosion and instability. Grade control structures are also often necessary to meet the maximum permissible velocity, presented in Section 4 *Open Channels*, for major design storm runoff in grass-lined channels and wetland channels. In some locations, such as those adjacent to schools or parks, it may be advisable to further reduce design velocities to diminish safety risks to children. Drop structures are used at locations where the use of channel lining materials is undesirable or does not sufficiently reduce design velocity in the channel (see Section 4 for the design of lined channels). A properly designed drop structure will effectively reduce the slope in a channel segment, safely dissipate energy produced by the drop, and reduce adverse erosive effects to the channel bed.

Control Sill Grade Control Structures, or Low-Flow Check Structures, are used for velocity and grade control in wide, relatively stable floodplains and wetland areas. These structures are addressed in Section 4.5.

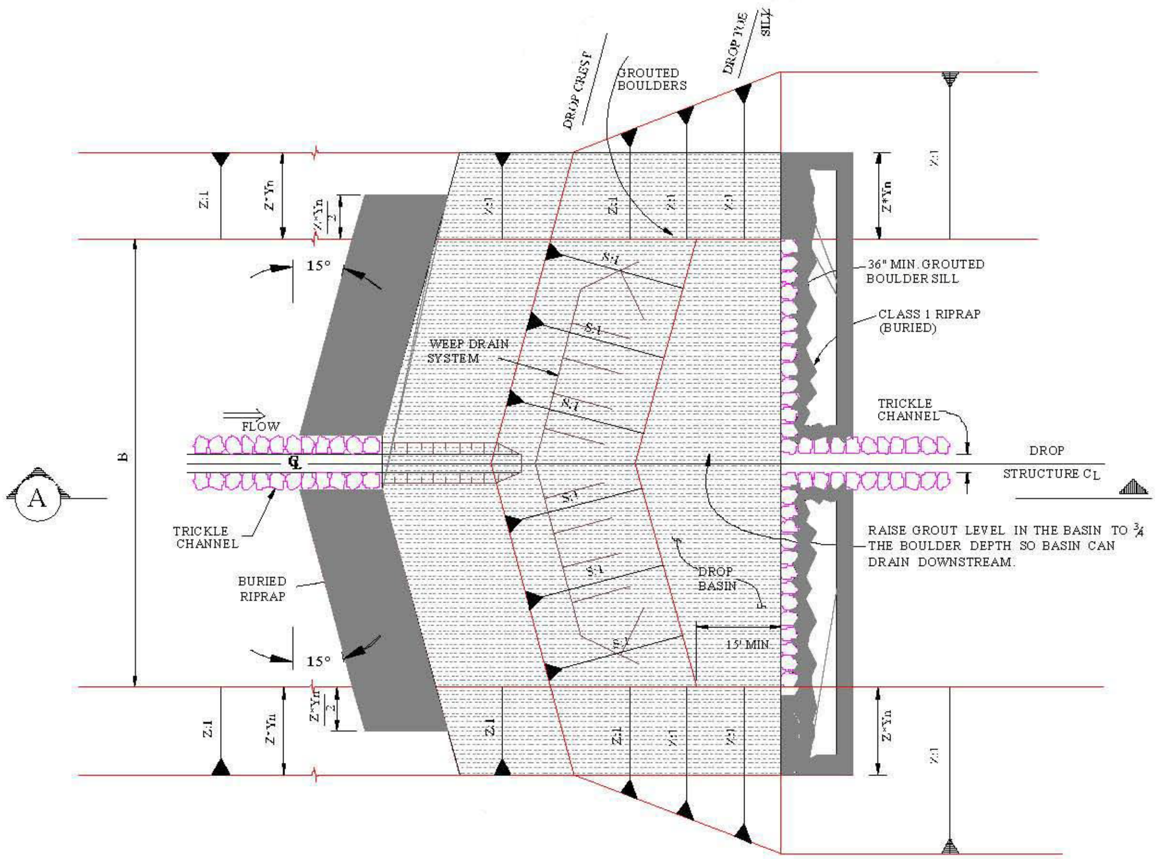
Grade control structures should be used to provide channel stability and should be limited to a drop height of three (3) feet. Four types of grade control structures are addressed in this section: 1) grouted sloping boulder (GSB) shown in Figure 9-1, 2) vertical hard basin (VHB) shown in Figure 9-2, 3) baffle chute drops shown in Figure 9-3, and sculptured concrete (SC) drop structures shown in Figure 9-4. GSB drop structures are preferred; however, limited availability of materials may require concrete vertical drops with riprap basins or other materials to be used. For larger single drops, the straight drop spillway, the baffled apron, or one of the structures found in Section 9.4 may be employed. Generally, the designer is referred to information and design guidelines presented in the Urban Storm Drainage Criteria Manual (USDCM), Volume 2, "Hydraulic Structures" for an extensive discussion of the design of drop structures (http://udfcd.org/downloads/down_critmanual_volII.htm).

9.3.1 Grouted Sloping Boulder (GSB) Drop Structure

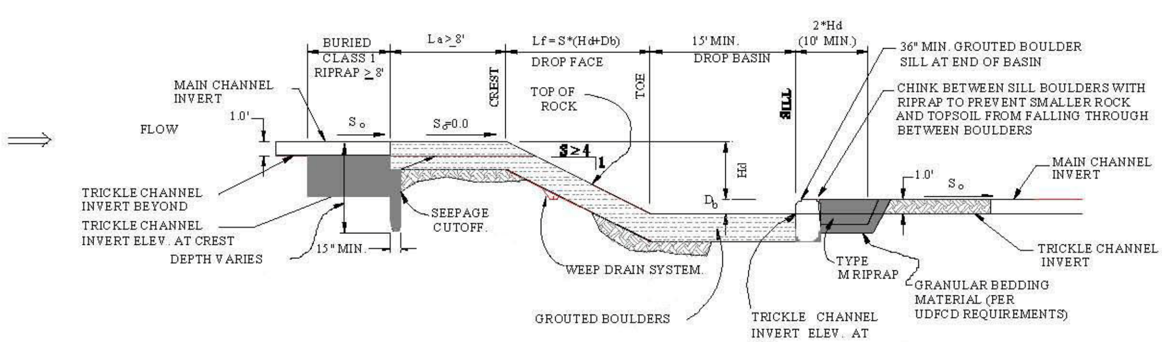
The GSB drop structure, shown in Figure 9-1, has recently become one of the more commonly installed drop structures in new construction and channel retrofit situations because of relatively good hydraulic effectiveness and generally pleasing aesthetics. However, local availability of rock that meets the size and quality requirements for this structure weighs heavily on the economic viability of GSB drops. It is intended for use only in grass-lined channels with upstream velocities within the limits set forth in Section 4 of these Drainage Criteria. With some variation in design, GSB structures may be used with channels containing or not containing a trickle channel or a low-flow channel.

The excess energy created by the invert drop is dissipated in two ways in a GSB structure. The additional roughness of the grouted boulders themselves is secondary in energy dissipation to the hydraulic jump formed in the drop basin downstream. However, improper design or construction of the grouted boulders, including faulty rock or grout selection and placement, may result in sweep out of the hydraulic jump due to excessive velocity in the drop basin.

Figure 9-1
Grouted Sloping Boulder Drop with Trickle Channel



DROP STRUCTURE PLAN
NTS



DROP STRUCTURE PROFILE (A)
NTS

Rock and Grout

Standard riprap rock gradation is not utilized in GSB structures. Instead, boulders are placed in one layer directly on the graded and properly compacted subgrade, as close together as possible and so as not to adversely disturb the subgrade. Boulder sizing is based upon the critical velocity, V_{c1}, in the channel upstream from the drop structure. If a trickle channel or low-flow channel is present, the maximum critical velocity of that channel and the main channel is used to find the rock-sizing parameter using Equation 9.1:

$$R_p = V_{c1} S^{0.17} / (G_{s,rock} - 1)^{0.66} \quad (9.1)$$

Where:

R_p = Rock-sizing parameter
V_{c1} = Maximum upstream critical velocity (fps)
S = Longitudinal slope (ft/ft)
$G_{s,rock}$ = Specific gravity of rock (2.55 unless otherwise certified by quarry)

This parameter is used in Table 9-2 to find minimum boulder dimension, D_r.

Grouted boulders must be placed upstream of, and along, the crest of the drop, in the drop face and basin, and along the sill at the end of the basin. The flattest surface of each boulder is oriented upward and as close to horizontal as possible. The boulders shall be placed such that breakage or spalls are minimized. The boulders shall be checked for significant cracking before grouting and shall be cleaned with water before grouting to improve grout-rock adherence. Damaged boulders shall be replaced before grouting.

Table 9-2
Grouted Sloping Boulder (GSB) Drop Structure Rock Sizing

Rock Sizing Parameter, R_p	Minimum Boulder Dimension, D_r (inches)
Less than 4.50	18
4.50 – 4.99	24
5.00 – 5.59	60
5.60 – 6.39	36
6.40 – 6.99	42
7.00 – 7.50	48

Source: USDCM (UDFCD) Table HS-5.

Grout is used to fill the voids between the boulders from the subgrade up to one half of the boulder height from the subgrade. In the drop basin only, this is increased to three quarters of the boulder height from the subgrade to promote draining. Excessive grouting may cause a reduction in hydraulic capacity and energy dissipation, and may endanger the structural stability of the drop. Selection, mixing, placement, and finishing of grout shall comply with Section 02273, Riprap, of the City of Gillette Standard Construction Specifications.

Upstream Channel and Approach Apron

Grouted boulders shall be placed on grade with the upstream channel for a minimum of 8.0 feet upstream from the drop crest (this shall be referenced as the "approach apron"). Buried riprap shall be installed from the upstream end of the approach apron to a point at least 8.0 feet upstream along the channel thalweg. The riprap shall be D_{50} = 12 inches and shall be installed as described in Section 4.4.9 of these Drainage Criteria. The grouted boulder approach apron shall be continuous across the width of the channel (except as described in the following paragraph) and up each bank to the elevation of the normal depth for the design flow at that location. The buried riprap shall be installed across the channel bottom and up each bank to the elevation of one half the normal depth for the design flow at that location.

For grass-lined channels with a concrete or rock-lined trickle channel (see Section 4.4.7 of these Drainage Criteria), the approach apron and upstream riprap protection are discontinuous across the channel cross-section to allow the trickle channel flowline to continue unimpeded to the drop crest. While this is necessary to retain the effectiveness of the trickle channel in conveying base or nuisance flows, it tends to create a concentrated jet at the location of the trickle channel during higher flow periods. The additional energy introduced to the basin in these cases may be partially dissipated by the installation of large boulders or baffles in the trickle channel and/or a meandering trickle channel through the drop basin itself.

Grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material because of the high potential for seepage and piping under and around the drop structure. Since the GSB structure is rigid and essentially monolithic, seepage under the grouted boulders and the resultant transport of subgrade particles will eventually lead to structural failure. Therefore, a seepage cutoff section is required as shown in Figure HS-7A in the Urban Storm Drainage Criteria Manual (USDCM), Volume 2, "Hydraulic Structures". The dimensions of the vertical cutoff shall be determined based on geotechnical investigations and seepage analysis or shall comply with the minimum cutoff criteria set forth in the appropriate figures. The seepage cutoff shall be installed prior to the placement of the grouted boulders at the drop crest, and shall include a keyway for the grout/cutoff interface as shown in the details.

Drop Face

The drop face shall consist of grouted boulder "steps" of vertical dimension no greater than one half of the minimum boulder dimension, D_r, from Table 9-2. The overall drop face slope must not exceed 4(H):1(V); flatter slopes are encouraged due to improved aesthetics and energy dissipation. Steeper slopes may reduce structural stability.

The grouted boulders are continuous across the entire bottom width on the drop face. The grouted boulders also continue up each bank to the elevation equivalent to the downstream channel normal depth (sequent subcritical depth) plus freeboard or the channel critical depth plus 1.0 feet, whichever is greater.

A weep drain system shall be installed behind the drop face to relieve hydrostatic pressure in drops exceeding 5.0 vertical feet. See details in Figure HS-7A in the Urban Storm Drainage Criteria Manual (USDCM), Volume 2, "Hydraulic Structures".

Drop Basin

The basin area shall be constructed of continuous grouted boulders of the same dimensions as the drop face section (boulder size, crest and basin width, height of bank protection). However, the grout level is increased to three quarters of the boulder height in the basin, and shall be sloped to drain to the centerline of the channel (or trickle channel if applicable).

The basin is depressed below the downstream channel invert by 2.0 feet to help stabilize the hydraulic jump. Basin length shall be a minimum of 15 feet for non-flexible downstream channel lining (concrete, grouted riprap, geosynthetic linings) and a minimum of 20 feet for downstream channels with flexible linings. A row of 36-inch or larger grouted boulders shall be placed at the downstream end of the basin. The top of this sill shall be equal to the invert of the downstream channel. For channels with a concrete or rock-lined trickle channel, there shall be a break in the end sill of width equal to that of the trickle channel. The trickle channel shall continue downstream through the sill and exit apron with scour protection as specified in Section 4 of these Drainage Criteria.

Exit Apron

The exit apron shall consist of buried riprap of size D_{50} = 12 inches and shall be installed per the criteria described in Section 4.4.9 of these Drainage Criteria. The riprap shall extend across the channel (except in the trickle channel as applicable) and up the banks to an elevation equal to the top of the adjacent grouted boulders. This riprap protection shall extend downstream from the end sill a minimum distance of twice the drop height or 10 feet, whichever is greater.

9.3.2 Vertical Hard Basin Drop Structure (VHB)

The VHB drop structure, shown in Figure 9-2, consists of an approach apron (grouted rock), a vertical crest wall (concrete), a stilling basin with end sill (grouted rock or concrete), and downstream channel scour protection. While effective for energy dissipation, these structures shall be avoided if possible in areas of significant public use or in highly visible locations due to safety concerns and low aesthetic appeal. The maximum allowable drop for a VHB drop structure of this type shall be 3.0 feet.

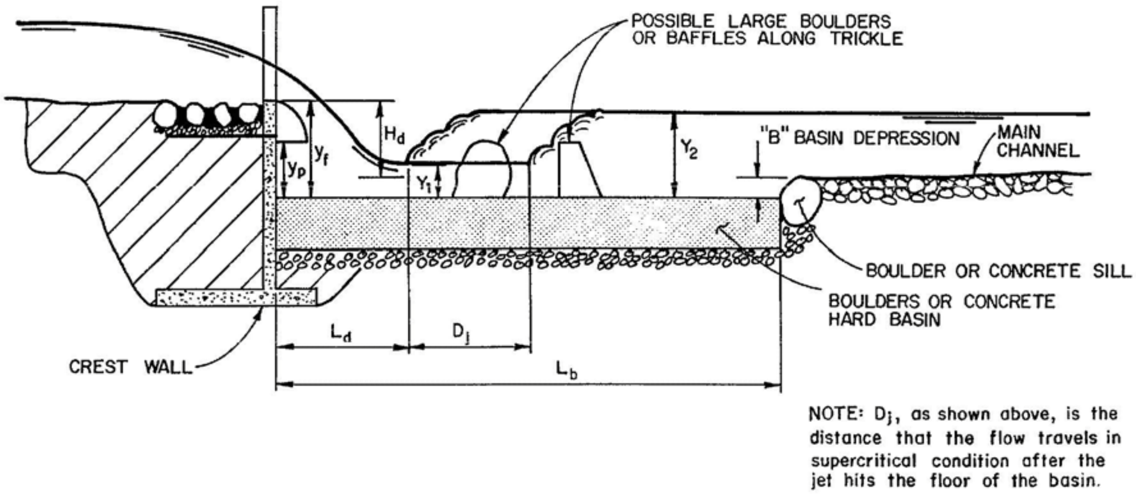
Rock and Grout

Rock used upstream of the crest wall shall have a minimum dimension of 12 inches in any direction. Rock used downstream of the drop shall have a minimum dimension of 18 inches in any direction. Grouting requirements are identical to those presented in Section 9.3.1 for the GSB drop structure.

Approach Apron

A grouted rock apron shall be installed across the entire bottom width (including trickle and low-flow channels) and up each bank to the elevation equal to upstream channel normal depth plus 1.0 feet. The rock shall be buried to a depth such that the top of the grout is equal to the invert of the upstream channel at every point across the channel. This approach apron shall extend upstream from the crest wall a minimum of 10 feet.

Figure 9-2
Vertical Hard Basin Drop



Vertical Crest Wall

The crest wall is a retaining wall that shall be designed by a structural engineer. Dimensions shall conform to the upstream inverts for the trickle or low-flow channel and the main channel across the bottom width. The wall shall extend a minimum of 5.0 feet into the undisturbed banks. However, all design dimensions including minimum structural width, wall thickness, footer size and geometry, and reinforcement shall be determined using accepted structural analysis methods and determination of potential creep, heave, buoyancy and uplift due to seepage pressures, and all other considerations associated with the design of a retaining wall.

Impervious backfill material is recommended both upstream and downstream adjacent to the crest wall and footers to act as a horizontal seepage cutoff. Other means may be employed to ensure minimized seepage around/under the crest wall. Piping, the transport of structural supporting material away from its intended location, is a common cause of structure instability and failure.

Basin

The basin is a depressed, hard-surface area which redirects the plunging flow from the crest horizontally. At lower flows, the energy dissipated by this redirection may be sufficient to return the flow to a subcritical state. However, the primary energy dissipation method for this structure is a hydraulic jump formed in the basin. When the upstream channel is composite (utilizing a trickle or low-flow channel), the approach velocity tends to be higher in the smaller sub-channel zone than in the main channel zone. Therefore, for the design flow, the basin length and downstream protection requirements may differ for the two zones. By placing large boulders (60% to 80% of critical depth in height) between the location of nappe impingement on the basin floor and a point at least 10 feet from the end sill, the required basin length for the sub-channel zone may be reduced to that of the main channel zone. Otherwise, the following calculations must be applied to both zones independently.

The drop is to be treated hydraulically as a straight-drop spillway and analyzed according to Chow (1959). A "drop number", D_N, must first be calculated in order to relate other associated lengths and depths:

$$D_N = q^2 / gh^3 \quad (9.2)$$

Where:

q = Discharge per unit width for the Subject Zone (cfs/ft)
g = Gravitational acceleration (32.2 ft/s^2)
h = Effective height of drop (ft)

Note that the effective drop height must include the basin depression depth. Using the drop number, the following relationships can be solved:

$$L_d/h = 4.3D_N^{0.27} \quad (9.3)$$

$$y_p/h = D_N^{0.22} \quad (9.4)$$

$$y_1/h = 0.54D_N^{0.425} \quad (9.5)$$

$$y_2/h = 1.66D_N^{0.27} \quad (9.6)$$

Where:

L_d = Drop length (ft)
y_p = Pool depth under nappe (ft)
y_1 = Depth upstream of the hydraulic jump (ft)
y_2 = Subcritical sequent depth (ft)

These values assume that atmospheric pressure is maintained under the nappe, thus the designer is responsible for incorporation of aeration devices as necessary. Drop length, L_d, refers to the horizontal distance from the crest wall to the location of depth y_1, upstream of the hydraulic jump. The basin design length, for the subject zone, is given by Equation 9.7:

$$L_b = L_d + D_j + 0.60L_j \quad (9.7)$$

Where:

L_b = Basin design length (ft)
D_j = Distance from location of Depth y_1 to jump (ft)
L_j = Length of Jump $\approx 6y_2$

The distance from the point of nappe impingement on the basin floor to the upstream end of the hydraulic jump is determined by a water surface profile analysis.

Basin depression depth below the downstream channel invert is determined by comparing the subcritical sequent depth, y_2, with the tailwater depth in the downstream channel, y_{TW}. If y_2 exceeds y_{TW}, the jump will be swept downstream and possibly out of the basin. This situation is to be avoided since significant erosion may take place if the jump occurs in an unarmored location in the channel. If y_{TW} exceeds y_2, the jump is pushed upstream toward the wall, potentially submerging jump, which may affect the structural design of the crest wall. Basin depression effectively adds to the tailwater depth in the downstream channel, controlling the location of jump formation. Therefore, the minimum basin depression depth, B, is:

$$B = \text{MAX}\,[1.5,\ (y_2 - y_{TW})]\ (\text{ft}) \tag{9.8}$$

This is the height of the end sill and downstream invert above the downstream end of the depressed basin. The end sill shall be constructed of reinforced concrete or grouted boulders of a minimum 36-inch dimension. This acts as a protected transition back to the channel invert.

Downstream Channel Protection

The channel directly downstream from the end sill shall be protected for a minimum of 10 feet in the direction of flow with buried riprap of size D_{50} = 12 inches or grouted rock with a minimum dimension of 12 inches.

In cases where the sub-channel zone basin length is longer than the main channel zone (no additional boulders or baffles placed in the basin to dissipate the center jet), the additional protection shall extend a lateral distance equal to the bottom width of the trickle channel from each edge of the trickle channel. This results in an extended protection zone with a width equal to three times the trickle channel bottom width.

9.3.3 Baffled Aprons (USBR Type IX, Baffle Chute Drop)

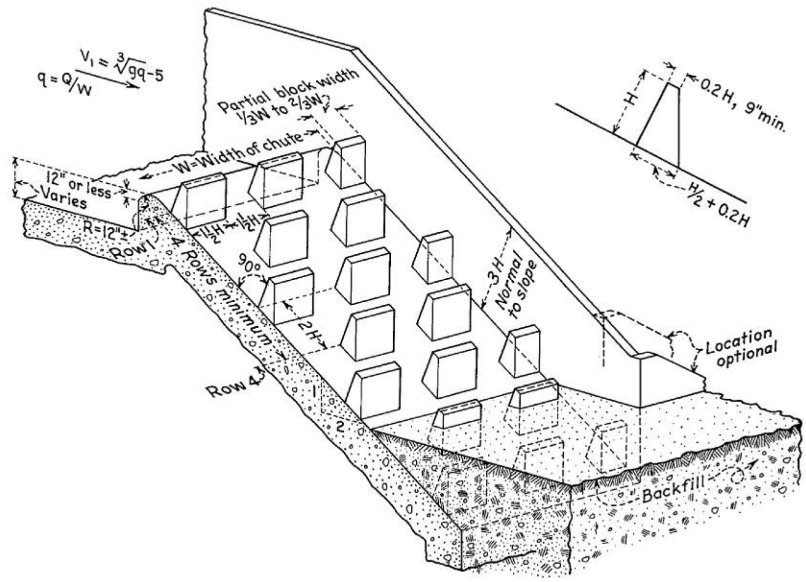
The costs associated with the construction of a baffled apron structure (hereafter referred to as a baffle chute drop) typically limit their use to larger drops from an economic standpoint, although the actual minimum size is limited to that length required to incorporate the minimum number of baffle rows. These drop structures are most effective at unit discharge rates between 35 and 60 cfs per linear foot. However, a value in this range can often be attained by altering the width of the chute. Most often, transition walls are employed to direct wider upstream channel flow to a narrower chute, decreasing the cost of the drop structure. When designed and built correctly, these structures are effective and last for many years with minimal maintenance requirements.

While the baffle chute drop can pass most sediment and debris, larger debris may become caught behind the baffles or in the narrowed chute, disabling the structure's ability to dissipate energy. This can lead to an effectively higher invert in the chute and overtopping, and can also allow the nearly unimpeded flow in the chute to exit the structure at erosive velocities. Therefore, debris-control structures are recommended upstream of the drop, and regular inspection and maintenance may be necessary.

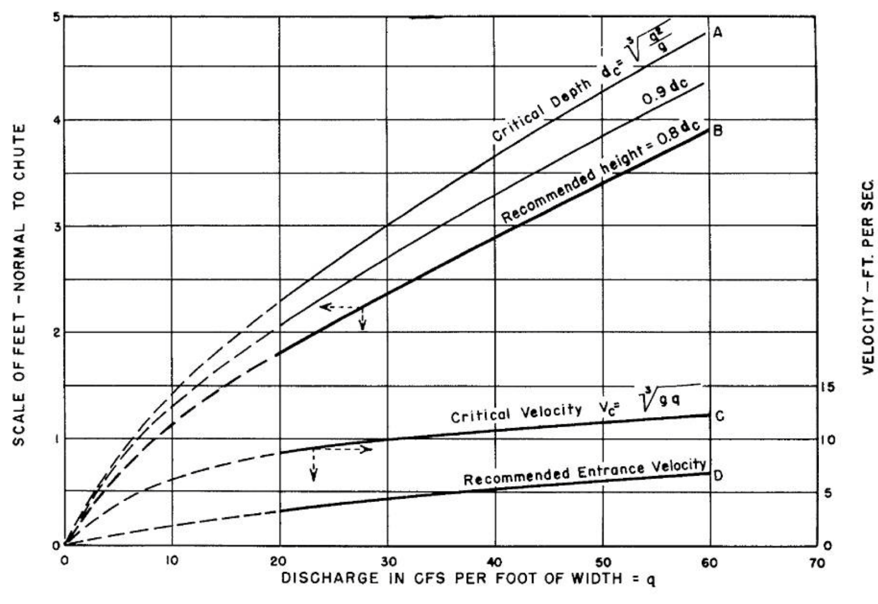
The baffle chute drop structure does not rely on the formation of a hydraulic jump as its primary energy dissipation process. Instead, excess energy in the chute flow is dissipated by redirection over and around baffle blocks, which are arranged in offset rows to avoid the passing of high-velocity jets between the blocks. Since a hydraulic jump is not part of the design, there are no tailwater requirements for this structure. However, potential scour due to relatively high velocities at the end of the chute and in the downstream transition section necessitate a protected exit apron and/or scour hole.

Figure 9-3 presents an isometric view of a baffle chute drop with typical dimensional requirements. Note that this figure does not indicate structural requirements such as concrete thickness or reinforcement, footer depths and dimensions, or seepage control. These factors shall be assessed and approved by qualified professionals.

Figure 9-3
Baffled Chute Drop



(A) USBR ISOMETRIC



(B) DESIGN CRITERIA

Upstream Channel Transition

Typically, the design width of the baffle chute drop is less than the upstream channel width for economic and sizing reasons as well as to attain unit discharge rates in the desired range. The headwalls and/or wingwalls associated with this transition are subject to design constraints set forth in these Drainage Criteria and shall be designed using proper structural analysis techniques. The designer should note that the effective width of a conduit or channel is often considerably smaller than the physical width due to the separation of flow from the abutment/conduit interface.

The approach section downstream from the transition is designed to maintain an approach velocity of less than the critical velocity at the crest. Recommended approach velocities are presented in Figure 9-3. The concrete flow alignment apron, reaching from the abutment/conduit interface to the chute crest, shall be a minimum of 5.0 feet in length and shall be equal to the chute in width along its entire length. In certain cases, the transition section may not sufficiently reduce the specific energy of the flow to achieve the proper approach (alignment) apron velocity. In these situations, the crest may be raised by up to 12 inches above the approach apron invert.

If a trickle channel is present in the upstream channel, it shall continue through the transition section and apron, and shall maintain a continuous flowline through any raised crest.

Transition and apron wall heights are determined by backwater analysis at peak flow, with a freeboard equal to or greater than that of the upstream channel.

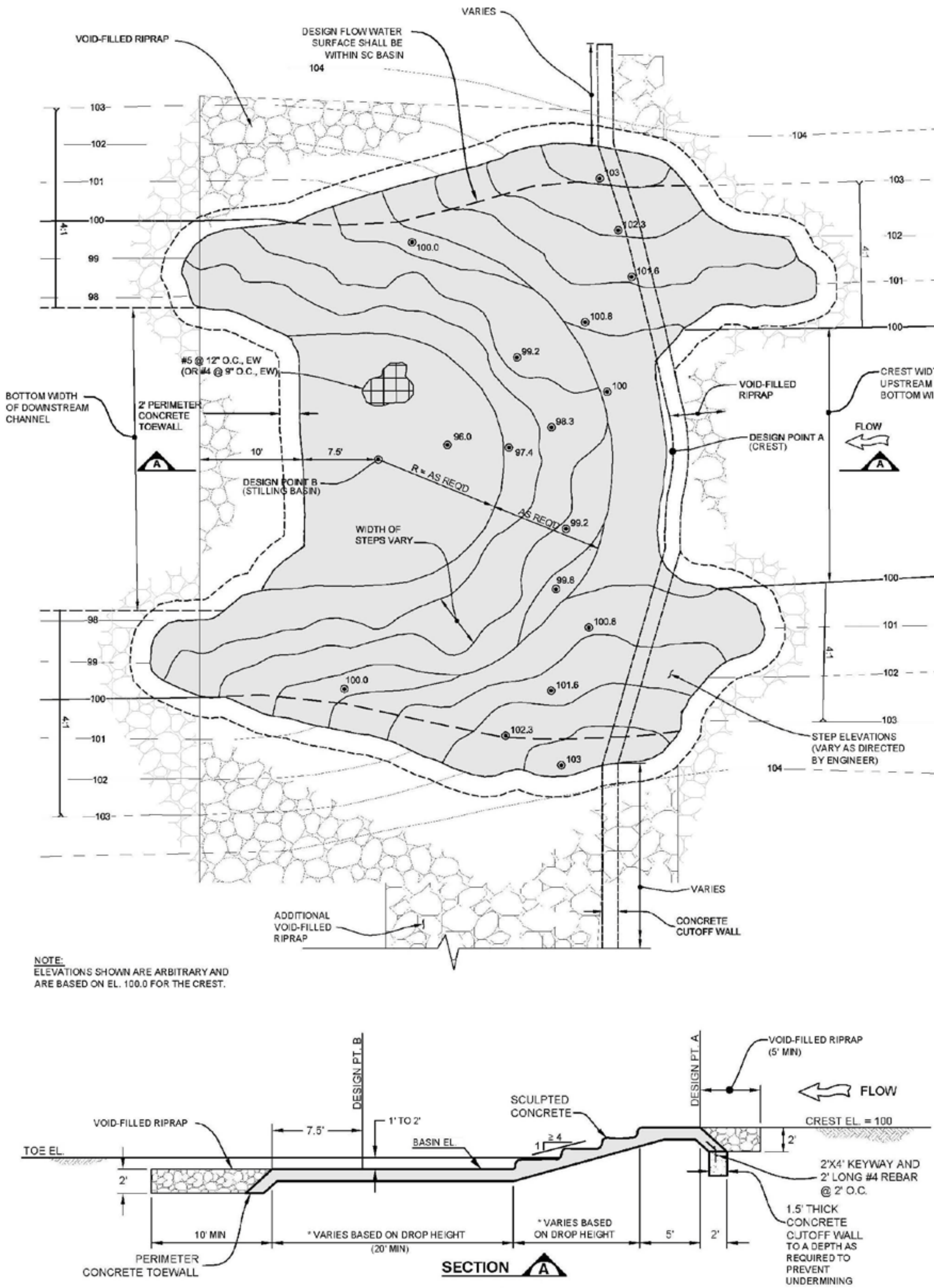
Baffled Chute

The chute floor, walls, and baffles shall be constructed of reinforced concrete and shall be structurally designed to withstand all geotechnical, hydrostatic, and hydrodynamic (impact and frictional) forces imposed by the specific site conditions, including a reasonably conservative factor of safety for all loading. The chute floor shall have a slope no steeper than 2(H):1(V) (Z:1, $Z_{MAX} = 2$). The chute walls shall be vertical and shall be tied to the floor, upstream wall or abutments, and downstream abutments with properly sized and installed steel reinforcement.

9.3.4 Sculptured Concrete (SC) Drop Structure

Sculptured Concrete (SC) drop structures, shown in Figure 9-4, can be used as an alternative to the GSB drop structure where local availability of rock makes the GSB uneconomical. When properly designed and constructed, the SC drop structure has good hydraulic effectiveness and pleasing aesthetics, and can be used in new construction and channel retrofit situations. Excess energy created by the invert drop is dissipated by the additional roughness of the sculptured concrete “steps” and by the hydraulic jump formed in the drop basin downstream. It should be used only in grass-lined channels with upstream velocities within the limits set forth in Section 4 of these Drainage Criteria. With some variation in design, SC structures may be used with channels that have a trickle channel or a low-flow channel.

Figure 9-4
Sculptured Concrete Drop Structure



The designer shall determine the design flow recurrence interval, which does not need to be the 100-year event. However, because the SC drop structure typically has no end sill to force a hydraulic jump, a complete hydraulic analysis documenting the performance and design of the drop must be performed to avoid a sweep out of the hydraulic jump due to excessive velocity in the drop basin. The designer must complete stability and scour analyses at the upstream and downstream limits of the structure. A range of design flow regimes, up to and including the major event, must be modeled to determine the stability of the drop and areas that need additional bank protection. Manning's 'n' for SC drops, assuming an approach channel depth of 5 feet or more, is 0.025. The value would be higher for lesser flow depths.

A geotechnical investigation and seepage analysis should be performed to obtain foundation design information, and to determine whether weep drains are required. Cutoff walls shall be designed to protect the structure from being undermined.

Drop Crest and Cutoff Wall

The crest of the SC drop structure is a flat area of concrete or shotcrete 5 feet in length and approximately equal in width to the bottom width of the approach channel, as shown in Figure 9-4. The location of the center of the crest is at Design Point A, which aligns with the center of the approach channel. The concrete toe wall shall be placed to a minimum depth of 2 feet along the upstream channel, and attached to a concrete cutoff wall designed to be below the potential scour depth. The crest shall follow the curvature of drop face as set from Design Point B. The cutoff wall shall separate from the toe wall at the edges of the approach channel and continue up the banks to the top of the shotcrete lining, which is placed generally on a line perpendicular to the centerline at the downstream edge of the crest. From the top of the shotcrete or concrete lining, the cutoff wall shall continue up the slope as required to be above the major storm water surface by a minimum of 1 foot.

Void filled riprap shall be installed for a minimum of 5 feet upstream of the cutoff wall in the bottom of the approach channel and above the edge of the shotcrete on the banks. The riprap shall be D_{50} = 12 inches minimum and shall be installed as described in Section 4.4.9 of these Drainage Criteria. The void filled riprap shall also be installed upstream of the cutoff wall above the top of the concrete.

For grass-lined channels with a low flow channel (see Section 4.4.7 of these Drainage Criteria), the sides of the low flow channel shall be flared and reduced in height to meet the shotcrete banks, to allow the low flow channel flowline to continue unimpeded to the drop crest. The riprap protection in the channel bottom would need to be extended upstream to protect the channel in this transition area.

The geometry of concrete steps on the banks upstream of the crest is up to the designer.

Drop Face

The drop face shall consist of concrete "steps" of vertical height no greater than 12 inches. The overall drop face slope must not exceed 4(H):1(V); flatter slopes are encouraged due to improved aesthetics and energy dissipation. The geometry of the drop face is laid out in a semi-circle based on a radius of ½ of the crest width (see Design Point B in Figure 9-4). The layout and grading of the sculptured concrete depends on the hydraulic performance and how the designer wants the structure to look. The designer should not make steps higher than 12 inches to protect public safety. The step length and heights do not need to be the same, and the structure does not need to be symmetrical.

The sculptured concrete shall continue up each bank to the elevation equivalent to the downstream channel normal depth of the design flow (sequent subcritical depth) plus freeboard or the channel critical depth plus 1.0 feet, whichever is greater. Void filled riprap shall be installed above the edge of the concrete basin on the banks to 1 foot above the water surface elevation of the major storm.

If warranted by the geotechnical conditions and seepage analysis, a weep drain system shall be installed beneath the drop face to relieve hydrostatic pressure. The seepage cutoff section is required as shown in Figure HS-7A in the Urban Storm Drainage Criteria Manual (USDCM), Volume 2, "Hydraulic Structures".

Drop Basin

The basin area shall be constructed of continuous concrete or shotcrete of the same bottom width as the downstream channel section. The basin is depressed below the downstream channel invert by 1.0 to 2.0 feet to help stabilize the hydraulic jump. Basin length shall be a minimum of 20 feet for downstream channels with grass linings.

The exit apron shall consist of buried riprap of size D_{50} = 12 inches and shall be installed per the criteria described in Section 4.4.9 of these Drainage Criteria. The riprap shall extend across the channel and up the banks to an elevation equal to the top of the adjacent riprap above the sculptured concrete. This riprap protection shall extend downstream from the end of the concrete a minimum distance of twice the drop height or 10 feet, whichever is greater.

Concrete and Shotcrete

Sculptured concrete may be constructed in one layer using a full thickness of shotcrete, or in 2 layers consisting of a substrate layer of structural concrete and a top layer of shotcrete. A minimum total thickness of 12 inches shall be maintained. Concrete and shotcrete shall be reinforced. Steps in the drop structure shall be formed by placing reinforcing bars horizontally across the drop face and on top of splice bars place vertically along the steps at 12 inches on center. Chicken wire shall be placed on the upslope side of the splice bars to hold the concrete at the step face during concrete placement.

9.4 ENERGY DISSIPATERS

All storm drains and culverts shall be designed with appropriate end treatments at their inlets and outlets, such as flared end sections or headwalls and wingwalls, and erosion protection or energy dissipaters shall be provided to limit erosion due to turbulent flow and high velocities. Energy dissipaters and erosion protection designs shall be based on the procedures described in the FHWA's *Hydraulic Design of Energy Dissipaters for Culverts and Channels* (HEC-14).

Where riprap is used for protection at inlet and outlets, the minimum size shall meet the requirements of the City Standard Construction Specifications, Section 02190.

9.5 BRIDGES

The decision to use a bridge rather than a large culvert should be made by estimating construction and maintenance costs, and considering structural, aesthetic, maintenance, operation, and environmental issues. Economic analysis of the design shall include complete life cycle costs and benefits.

The methodology of bridge design presented in the following paragraphs is intended for designers with a good understanding of basic hydrologic and hydraulic methods and with experience in the design of hydraulic structures. The designer must understand the variety of flow conditions which are possible in these complex hydraulic structures in order to make appropriate design decisions. A careful approach to bridge design is essential, both in new land development and retrofit situations, because bridges often significantly influence upstream and downstream flood risks, floodplain management, and public safety, and are generally more costly to build and maintain than large culverts.

9.5.1 Bridge Hydraulics

Bridges are required when roads or trails cross open channels and, therefore, sizing the bridge openings is of great importance. Open channels with improperly designed bridges can have excessive scour or deposition or inadequate conveyance for the design flow. Confining flood waters by bridges can cause excessive backwater resulting in flooding of upstream property, backwater damage suits, overtopping of roadways, costly maintenance, or even loss of the bridge. Bridge openings shall be designed to have as little effect on the flow characteristics as reasonable, consistent with good design and economics.

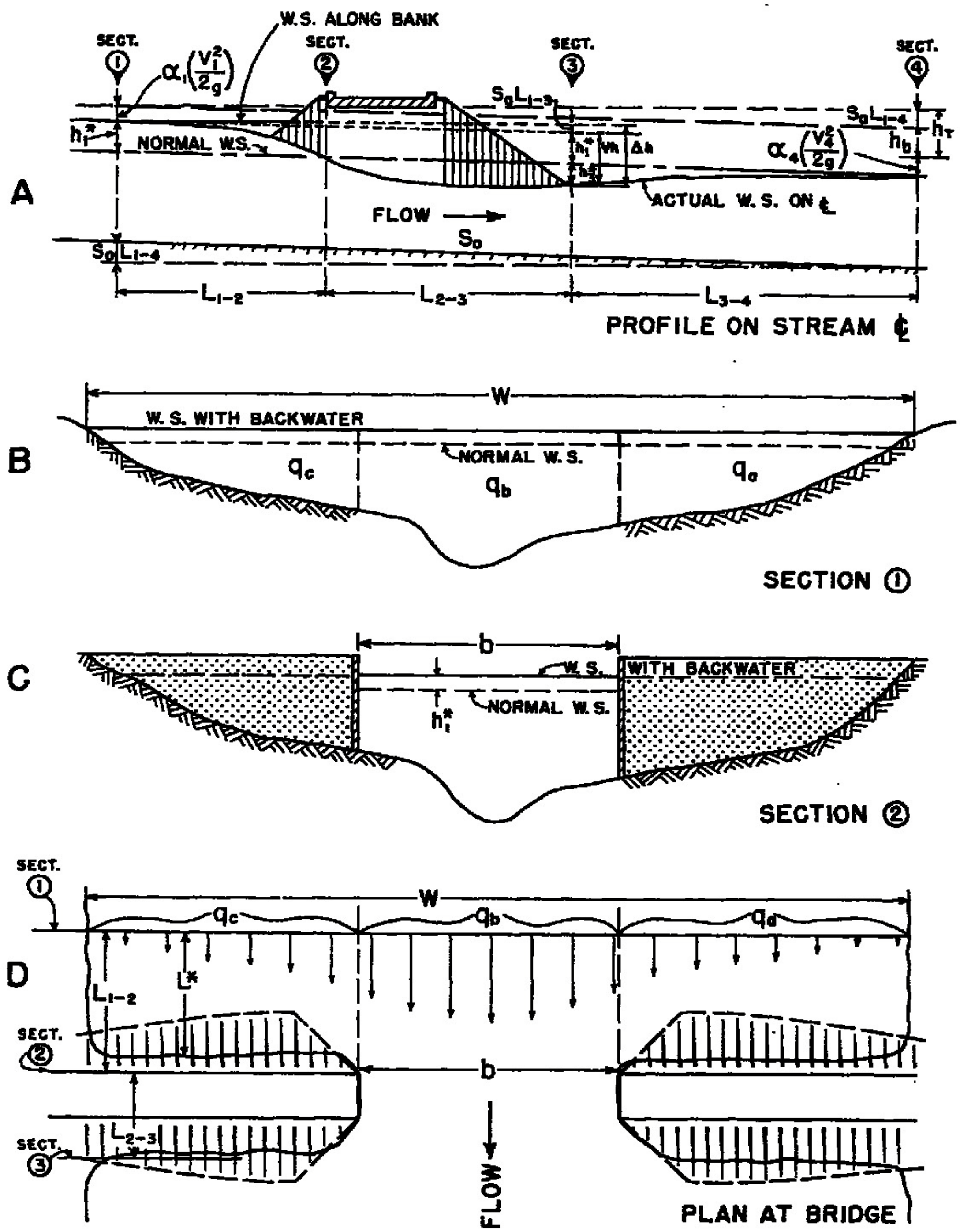
The hydraulic analysis procedures described in this Section are suitable, although methods such as WSPRO or HEC-RAS are acceptable as well. If a computer application other than WSPRO or HEC-RAS is to be used, the designer must first submit documentation of the program to the City for approval. The preliminary assessment approach described below is presented in greater detail in the FHWA HDS-1 *Hydraulics of Bridge Waterways.*

Backwater

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been reestablished, section 4 as shown in the "Profile on Stream Centerline" Figure 9-5. The expression is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical. The expression for computation of backwater upstream from the bridge constricting the flow is as follows:

$$h^{*}_{1} = K^{*}(v_{n2})^{2}/2g + \propto 1[(A_{n2}/A_{4})^{2} - (A_{n2}/A_{1})^{2}]\,(v_{n2})^{2}/2g \tag{9.9}$$

Figure 9-5
Normal Bridge Crossing Designation (HDS-1, FHWA)



Where:

h^*_1 = Total backwater (ft)
K^* = Total backwater coefficient
$\propto 1 = qv^2/QV_1^2$ = Kinetic energy coefficient
A_{n2} = Gross water area in constriction measured below normal stage (ft^2)
v_{n2} = Average velocity in constriction or Q/A_{n2} (ft/s). The velocity v_{n2} is not an actual measurable velocity but represents a reference velocity readily computed for both model and field structures.
A_4 = Water area at Section 4 where normal stage is reestablished (ft^2)
A_1 = Total water area at Section 1 including backwater (ft^2)
g = Gravitational acceleration (32.2 ft/s^2)
v = average velocity in a subsection (ft/s)
q = discharge in same subsection (ft^3/s)
Q = total discharge in river (ft^3/s)
V_1 = average velocity in river at section 1 or Q/A_1 (ft/s)

To compute backwater by this equation, it is necessary to obtain the approximate value of h^*_1 by using the first part of the equation, $h^*_1 = K^*(v_{n2})^2/2g$. The value of A_1, which depends on h^*_1, can then be determined. This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $(v_{n2})^2/2g$. The equation may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$

$V_{n2} < 7$ ft/s

$K^*(V_{n2})^2/2g < 0.5$

From Figure 9-5, $M = q_b/(q_a + q_b + q_c)$ or the bridge opening ratio, expressed as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the river.

If values meet all three conditions, the backwater obtained from the previous equation can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is required to use the equation in its entirety. The use of the guides is further demonstrated in the examples given in FHWA HDS-1 *Hydraulics of Bridge Waterways* that shall be used in all bridge design work.

The value of the overall backwater coefficient K^*, which was determined experimentally, varies with:

1. Stream constriction as measured by the bridge opening ratio, M
2. Type of bridge abutment: wingwall or spill through
3. Number, size, shape, and orientation of piers in the constriction
4. Eccentricity or asymmetric position of bridge with respect to the floodplain

5. Skew (bridge crosses floodplain at other than 90 degree angle)

The overall backwater coefficient K^* consists of a base curve coefficient, K_b, to which are added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K^* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

Figure 9-6 shows the base curves for backwater coefficient, K_b, plotted with respect to the opening ratio, M, for several wingwall abutments and vertical wall type. Note how the coefficient K_b increases with channel constriction. The several curves represent different angles of wingwalls as can be identified by the accompanying sketches. The lower curves represent the better hydraulic shapes.

Figure 9-6 shows the relation between the backwater coefficient, K_b, and M for spill-through abutments for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. Figures 9-6 and 9-7 are 'base curves' and K_b is referred to as the 'base curve coefficient.' The base curve coefficients apply to normal crossings for specific abutment shapes but do not include the effect of piers, eccentricity, or skew.

Figure 9-6
Base Curves for Wingwall Abutments (HDS-1, FHWA)

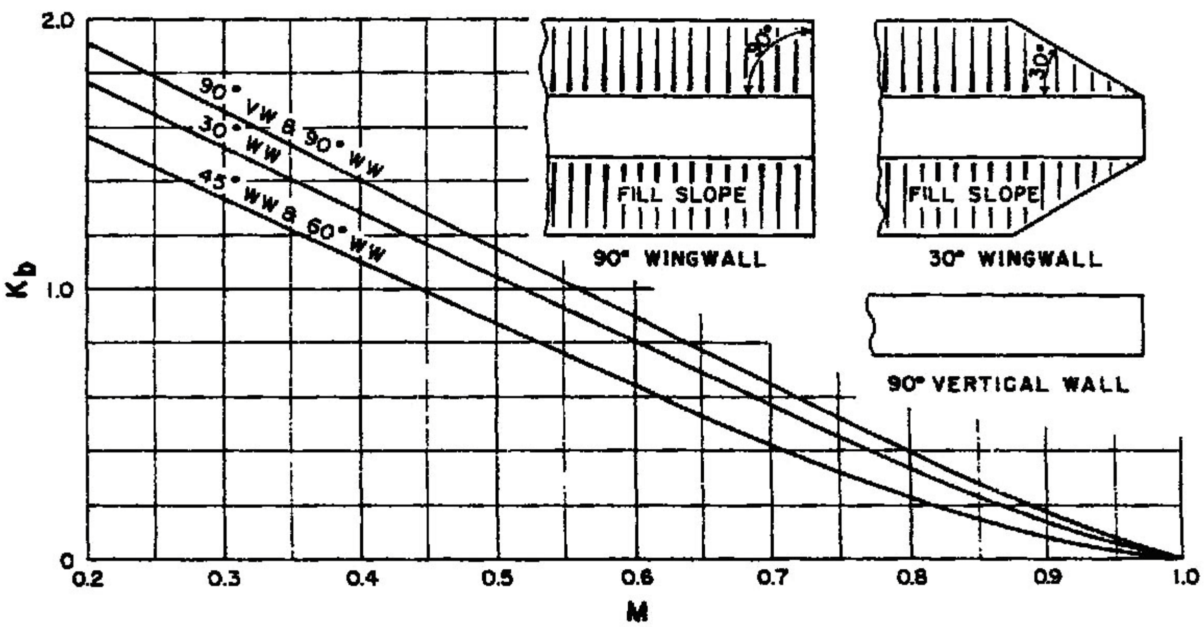
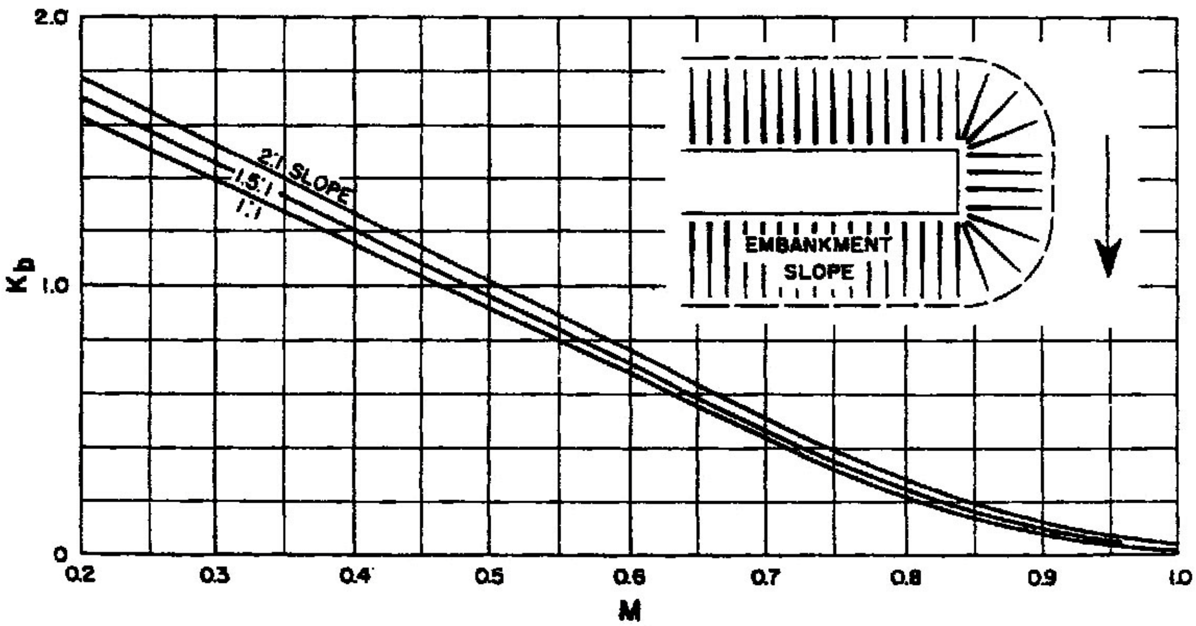


Figure 9-7
Base Curves for Spill Through Abutments (HDS-1, FHWA)



Effect of Piers

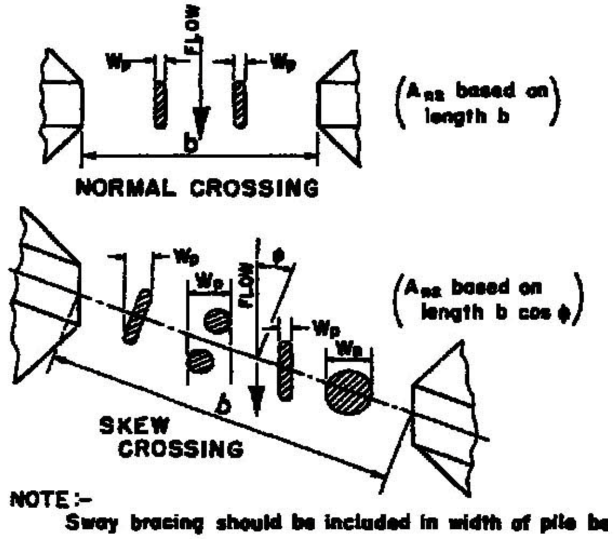
The effect of piers in a bridge constriction on backwater has been treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening; the type of piers (or pilings in the case of pile bents); the value of the bridge opening ratio, M; and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, has been assigned the letter J. In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure 9-8. The procedure is to enter Chart A, Figure 9-8, with the proper value of J and read ΔK and obtain the correction factor σ from Chart B, Figure 9-8, for opening ratios other than unity. The incremental backwater coefficient is then:

$$\Delta K_p = \Delta K \sigma \tag{9.10}$$

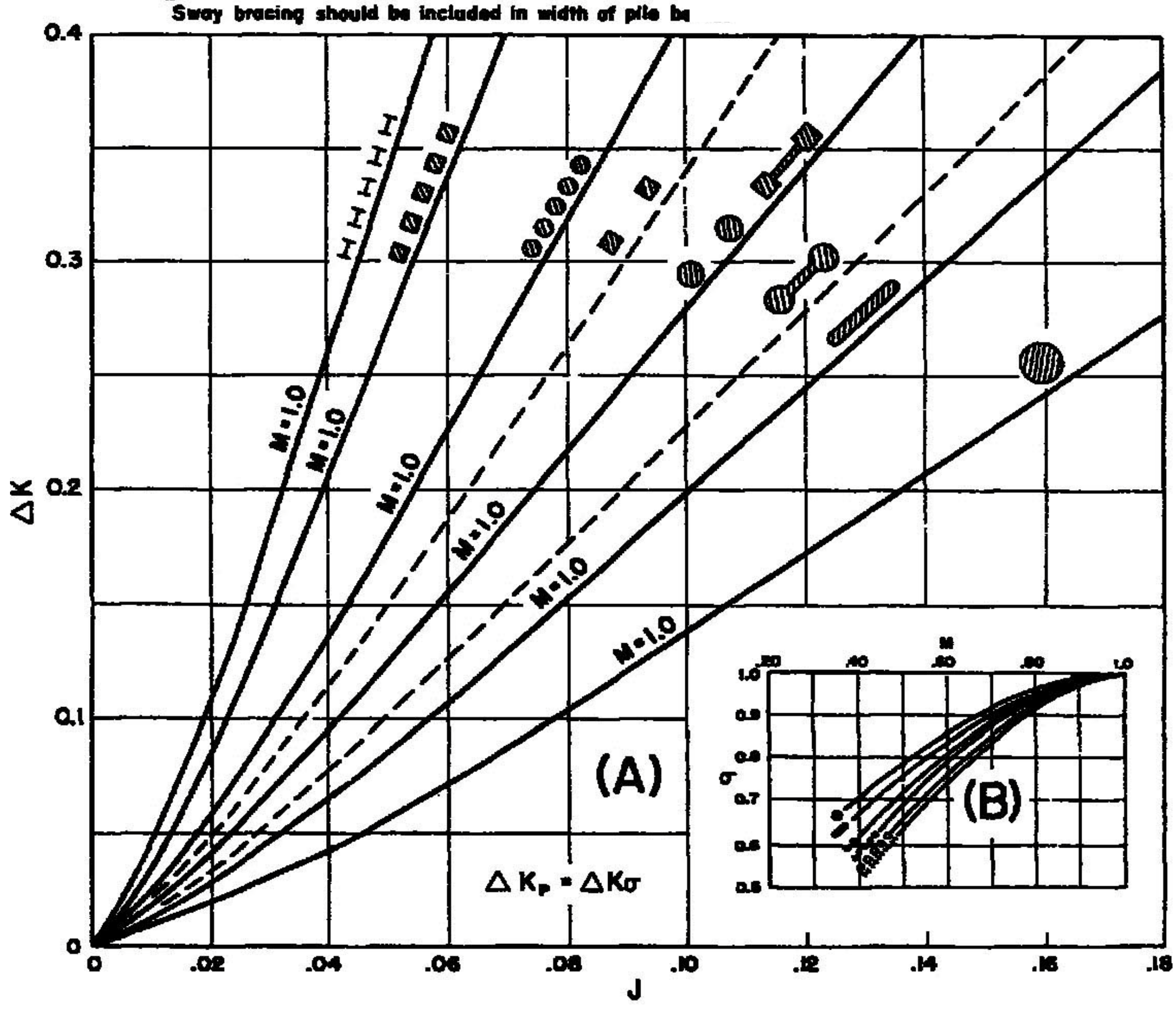
The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing but shall be increased if there are more than 5 piles in a bent. A bent with 10 piles shall be given a value of ΔK_p about 20 percent higher than those shown for bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figures 9-5 or 9-6) } + \Delta K_p \text{ (Figure 9-8)} \tag{9.11}$$

Figure 9-8
Incremental Backwater Coefficient for Piers (HDS-1, FHWA)



W_p	=	Width of pier normal to flow – feet
h_{n2}	=	Height of pier exposed to flow – feet
N	=	Number of piers
A_p	=	$\sum^N W_p h_{nt}$ = total projected area of piers normal to flow - square feet
A_{n2}	=	Gross water cross section in constriction based on normal water surfaces. (Use projected bridge length normal to flow for skew crossings)
J	=	$\frac{A_p}{A_{n2}}$



Bridge Hydraulic Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.
2. Determine the stage of the stream at the bridge site for the design discharge.
3. Plot representative cross section of stream for design discharge at Section 1. If the stream is essentially straight and the cross section substantially uniform in the vicinity of the bridge, the natural cross section of the stream at the bridge site may be used for this purpose.
4. Subdivide the above cross section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient, *n*, to each subsection. Typical roughness coefficients can be found in Table 4-1 of Section 4, *Open Channels*. Careful judgment is necessary in selecting these values.
5. Compute conveyance and then discharge in each subsection.
6. Determine the value of the kinetic energy coefficient.
7. Plot the natural cross section under the proposed bridge based on normal water for design discharge and compute the gross water area, including area occupied by piers.
8. Compute the bridge opening ratio, M, observing modified procedure for skewed crossings.
9. Obtain the value of K_b from the appropriate base curve.
10. If piers are involved, compute the value of J and obtain the incremental coefficient, ΔK_p.
11. If eccentricity is severe, compute the value of eccentricity and obtain the incremental coefficient, ΔK_e using methods in HDS-1.
12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain the incremental coefficient, ΔK_s, for proper abutment type using methods in HDS-1.
13. Determine the total backwater coefficient, K^*, by adding incremental coefficients to the base curve coefficient, K_b.
14. Compute the backwater by Equation 9.9.
15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in FHWA *Hydraulics of Bridge Waterways.*

9.5.2 Bridge Design Criteria

The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the Major storm event. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge shall be determined.

Bridge Design Standards

Bridges shall be designed to pass flows in accordance with the State of Wyoming Department of Transportation (WYDOT) Road Design Manual or the Design Guide for County Roads requirements. Bridges must be designed to pass flows so that upstream and downstream channels are not adversely affected and can continue to function as intended. The design and supporting calculations for both public and private bridges shall be prepared and certified by a Professional Engineer licensed in the State of Wyoming, and shall address at a minimum:

1. <u>Allowable rise</u> – The final design selection should consider the maximum backwater allowed by the National Flood Insurance Program (NFIP) unless exceeding the limit can be justified by special hydraulic conditions. For sites outside of Federal Emergency Management Agency (FEMA) regulation, the backwater shall not cause increased flood damage to property upstream of the crossing. The final design should not significantly alter the existing flow distribution in the floodplain, and velocities through the structure(s) shall not damage the roadway facility or increase damage potential to adjacent property.
2. <u>Design flows</u> – Bridges shall be designed to pass the Major storm event according to the classification of the roadway, see Table 8-1. The design capacity of any bridge will be the flow that will pass through the bridge opening with adequate freeboard and no roadway overtopping.
3. <u>Freeboard</u> – Clearance or freeboard should be provided between the low girder and the design water surface to allow for the passage of ice and debris; clearance between the lowest portion of the bridge's superstructure and the water surface elevation during the design flood shall be two (2) feet.
4. <u>Substructure</u> – Where bridge abutments and foundations are located below the 100-year water surface elevation, concrete wingwalls at angles of 40 degrees to 60 degrees shall be tied to the existing side slopes to prevent erosion behind the abutments. Pier spacing and orientation, and abutment location shall be designed to minimize flow disruption and potential scour. Bridge piers should not be placed in the main channel area.
5. <u>Roadway Geometry</u> – The "crest-vertical curve profile" is the preferred roadway bridge crossing profile when allowing for embankment overtopping at a lower discharge and for adequate deck drainage. Sag vertical curves can cause deck drainage to pond and ice up on the bridges and should be avoided. Horizontal curve transitions cause water to flow across lanes and should not be located on a bridge because of icing and hydroplaning problems.
6. <u>Supercritical flow</u> – When supercritical flow exists in a lined channel, the bridge shall have no influence on the flow.
7. <u>Scour</u> – Estimates of local and long term scour shall be calculated according to methods approved by WYDOT to determine the required bank protection and establish the depth of the bridge support structures. Estimate all degradation and aggradation plus contraction scour and local scour for the design year and for the 500-year event per the general design procedure in Section 2.2 of FHWA HEC-18. Scour depths are to be estimated with consideration of the local geology. Indicate the total scour envelope with

a continuous line drawn such that the structural designer may adequately design substructure components.

8. Countermeasures – Foundation design and/or scour countermeasures shall be made to avoid failure by scour. It is now often more efficient for a designer to design a pier (and if necessary the superstructure) for increased stream loads due to debris. When two or more bridges are constructed in parallel over a channel, care should be taken to align the piers and to provide streamlined grading and protection for abutments. This abutment grading is to minimize expansion or contraction of flow between the two bridges.

9. Environmental – Disruption of ecosystems is to be minimized. Consideration shall be given to the preservation of valuable characteristics that are unique to the floodplain and stream. Environmental permits may be required.

10. Right-of-way – Adequate right-of-way shall be provided upstream and downstream of the structure for construction of grading, countermeasures and long term maintenance operations.

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SECTION TEN
DETENTION

10.1 INTRODUCTION

Detention basins store runoff and reduce peak discharges by allowing flow to be discharged later at a slower, controlled rate within a reasonable time.

Detention is beneficial in controlling flood peaks in an urbanized area. Use of detention includes local detention, such as in a neighborhood park or within a parking lot, and regional detention, such as a large open space or off-line detention facilities. Regional and local detention facilities and the City of Gillette criteria for design of detention and retention ponds are discussed in this Section. To provide maintenance more efficiently and at the same time provide more appealing and useful open space areas, the City encourages project designers to investigate the feasibility of creating larger, regional detention ponds, which would serve larger drainage areas, instead of small pocket detention cells that serve individual developments. In general, detention ponds should be designed as multi-use facilities that include parks and open space amenities, and that are accessible for maintenance.

This section only addresses the hydrologic and hydraulic design of detention facilities for the City, and only with regard to the drainage requirements of the City. These facilities must also be designed for safety and stability. The designer is referred to the guidance in the NRCS Pond Standard 378 (NRCS 2002) and in Design of Small Dams (USBR 1977) for the important structural and geotechnical considerations of embankment stability, embankment foundations, outlet works design, seepage control, and spillway design.

10.1.1 Detention versus Retention

Stormwater storage reservoirs essentially fit into one of two categories: detention or retention. The words “pond” and “basin” are used interchangeably when used in connection with detention and retention reservoirs. A detention basin or pond “detains” water temporarily, releasing water through an outlet pipe or channel by means of a weir, orifice, or pump. Because of its ability to release flow during inflow, the overall volume of storage required for a given storm event is reduced. An advantage of the detention basin is the positive means of outflow, resulting in fewer problems with long-term ponding.

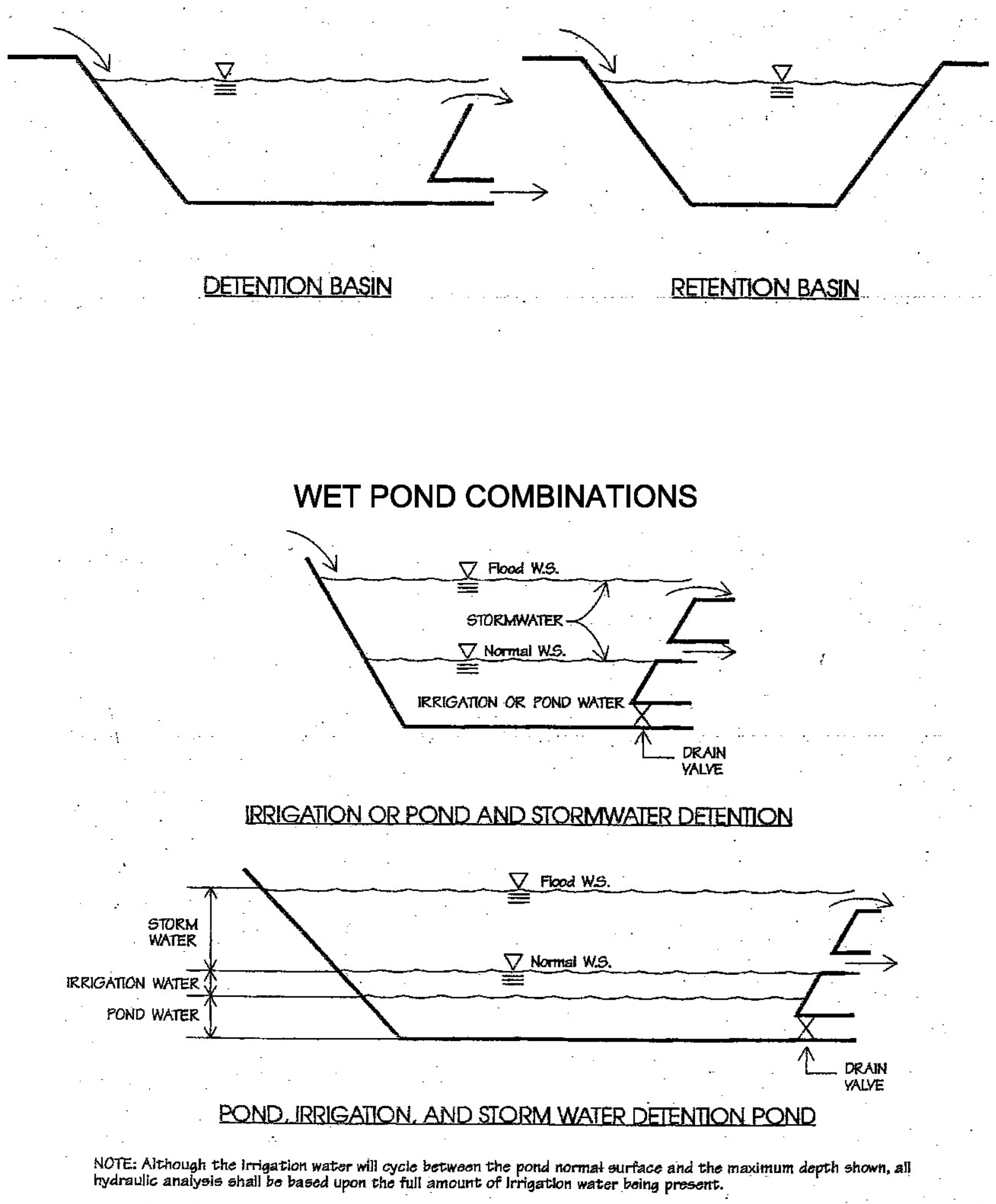
A retention basin or pond “retains” water without any initial release during inflow. Once the storm event is over, pond drainage occurs through evaporation and percolation into the soil. In some instances, retention basins may also involve a gated pipe or pump which is closed or inoperative during the storm event. However, if a gated pipe or pump is an available or desirable option, it would normally be advantageous to release water during stormwater inflow, which would change the basin from a retention basin to a detention basin. The difference in detention and retention basins is depicted in Figure 10-1.

The words “pond” and “basin” may be used to refer to reservoirs that are normally dry except during storm events, or store water for other purposes; e.g., irrigation and recreation, in addition to receiving stormwater during storm events.

Wet ponds are ponds where enough storage volume exists to provide normal recreational or irrigation water volumes below the required reservoir volume required for stormwater runoff.

Use of irrigation storage facilities for stormwater detention purposes is acceptable to the City; however, it is required that the stormwater reservoir volume provided be in addition to the maximum expected base storage (irrigation or wet pond) volume, as depicted in Figure 10-1.

Figure 10-1
Detention and Retention Basins



10.1.2 Local versus Regional Facilities

Local Minor Facilities

Local minor detention facilities are defined as serving a single development with a hydrologic watershed smaller than or equal to 20 acres and are designed to mitigate the impact of increased runoff due to development. The outlet capacity is generally based on predevelopment hydrology, and the detention structures are generally small (0.01 to 1 acre-foot). Detention storage volume may be provided as small landscaped or turfed basins, parking lot storage, or a combination of each. These facilities are typically privately owned and maintained, and often provide stormwater quality treatment.

Local Major Facilities

Local major detention facilities are defined as serving more than a single development or serving hydrologic watershed greater than 20 acres in size. These facilities may serve dual functions. They typically reduce existing flooding problems to allow more development and/or control increased runoff caused by additional development. They may handle both off-site and onsite flows. Due to their larger size, these basins are designed much the same as regional detention facilities. These facilities may be privately or publicly owned and maintained.

Regional Detention Facilities

Regional detention facilities are those identified in the City's Stormwater Master Plan or as designated by the City. Generally, these facilities control flow on major drainageways, serve multiple developments, and are owned and maintained by the City. The purpose of these facilities is to significantly reduce downstream flows in order to maximize the capacity of existing systems, maintain flows at or below pre-development rates and improve water quality.

10.2 STORAGE REQUIREMENTS

Since urban development increases the rate, volume, duration, and frequency of stormwater runoff, measures must be implemented to avoid harm to downstream properties. Detention is considered a viable method to reduce development impacts and drainage infrastructure costs. Temporarily detaining storm runoff can significantly reduce downstream flood hazards as well as reduce required pipe and channel sizes in developed areas. Temporary storage also provides for sediment and debris collection which helps to maintain water quality in downstream channels and streams. However, detention may not be necessary where downstream drainage facilities have adequate capacity to convey runoff from fully developed upstream areas without damage.

Detention is required for all new development or redevelopment to limit major and minor peak stormwater discharges to release at pre-development rates, if the capacity of the downstream drainage system will be exceeded by developed flows.

Storm runoff retention or over-detention may be used when downstream drainage facilities lack adequate conveyance capacity or are nonexistent, and construction of an outfall drainage system is impractical. Retention or over-detention may be used where there are severe limitations on the downstream conveyance capacity or where there is no outfall or drainage system to convey storm runoff from the development. The use of retention and over detention will be approved on a case-by-case basis by the City.

10.2.1 Areas without Master Drainage Plans

Criteria for design of on-site detention are provided in this section for a facility located in open space, parking lots, or underground. Underground detention is allowed only in commercial settings where redevelopment is taking place and when no other on-surface methods are practicable. In all cases, detention must be approved for use by the City. Retention ponds to capture 100-year runoff are discouraged and shall be considered only when there is no adequate drainageway or outfall available within a reasonable distance of the site.

10.2.2 Areas with Master Drainage Plans

In order for the City to consider allowing a new development to discharge to regional facilities, the following criteria must be met:

1. The City's Stormwater Master Plan recommends the regional detention facility.
2. The regional detention facility is designed to accommodate the fully developed flows from the upstream watershed that includes the site of the proposed development.
3. The regional detention facility is constructed, or will be constructed, in phases with the development; otherwise, temporary detention must be provided.
4. Legally-binding ownership and maintenance responsibilities are clearly defined to ensure the proper function of the facility in perpetuity.
5. There is adequate conveyance of the fully developed flows from the site to the regional detention basin.
6. Design is completed in accordance with these Drainage Criteria considering:
    a. Multi-use (e.g., recreation) shall be considered in the design of detention basins.
    b. The creation of jurisdictional dams (see 10.4.9) shall be strongly discouraged.
    c. Basins shall be located on existing publicly owned lands whenever possible.

10.2.3 Variances

At the sole discretion of the City, exemptions to on-site detention may be granted for the following conditions:

1. A proposed development which discharges to a regional drainage facility is exempt, provided the regional facility is completed in accordance with the City's Stormwater Master Plan, the regional facility was designed to include runoff from the proposed development, and there is adequate conveyance capacity for the drainage system from the development to the regional facility.
2. A proposed development that lies within the limits of a watershed wherein the City's Stormwater Master Plan explicitly exempts on-site detention for development.

3. A proposed development which discharges to an outfall drainage system that has adequate conveyance capacity for the 100-year flood from a fully developed watershed can be exempt.
4. A proposed development can be exempt when it can be demonstrated that the peak flows from the development will not increase the peak flows from the watershed for storm events up to the 100-year flood, as could be the case with LID techniques. The burden of proof is on the developer to demonstrate this condition.

Developers may be required to pay a fee-in-lieu of on-site detention. If this is the case, the City will contact the developer after application and provide specifics with regard to the fee amount and the purpose of the fee.

10.3 DESIGN CRITERIA

Detention shall be provided in dedicated areas of the site. Parking lots, open or landscaped areas or within drainage channels are acceptable locations. Rooftop detention shall not be allowed.

The presence of groundwater within detention ponds must be addressed in design. In general, ponds are designed to be either wet or dry. Depending on the geotechnical conditions a dry pond should have a depth to groundwater of 2 feet or more so that a stable surface is provided for maintenance. A wet pond should have a permanent water depth greater than 3 feet to reduce the potential to breed mosquitoes.

10.3.1 Volume and Release Rate

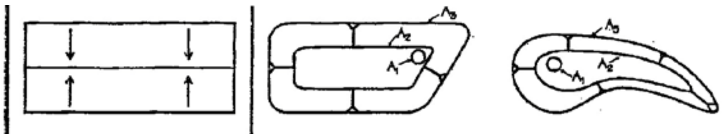
Detention storage shall be provided for all new development and redevelopment projects so that peak rates of runoff are reduced to rates less than or equal to undeveloped minor and major design storm rates, or shall be controlled to be consistent with the capacity of the downstream receiving system. For basins larger than 90 acres or when upstream detention ponds are present, detention ponds should be designed using the Unit Hydrograph [UH] Method. Detention ponds for basins smaller than 90 acres can be designed using simplified on-site detention sizing procedures. These methods are described in Section 10.3.3, Hydrologic Design Methods and Criteria.

Storage volume shall be calculated by the methods shown in Figure 10-2.

10.3.2 Design Frequency

Peak runoff from a site may not be increased in the minor and major design storms due to development. The site runoff may be a composite of detention/retention basin release/overflow and direct runoff, both of which must be considered. If direct runoff is allowed from the site, the sum of the direct runoff plus the release from the detention basin must not exceed the historic rate. This is depicted in Figure 10-3 and discussed further in Section 10.3.3, Hydrologic Design Methods and Criteria.

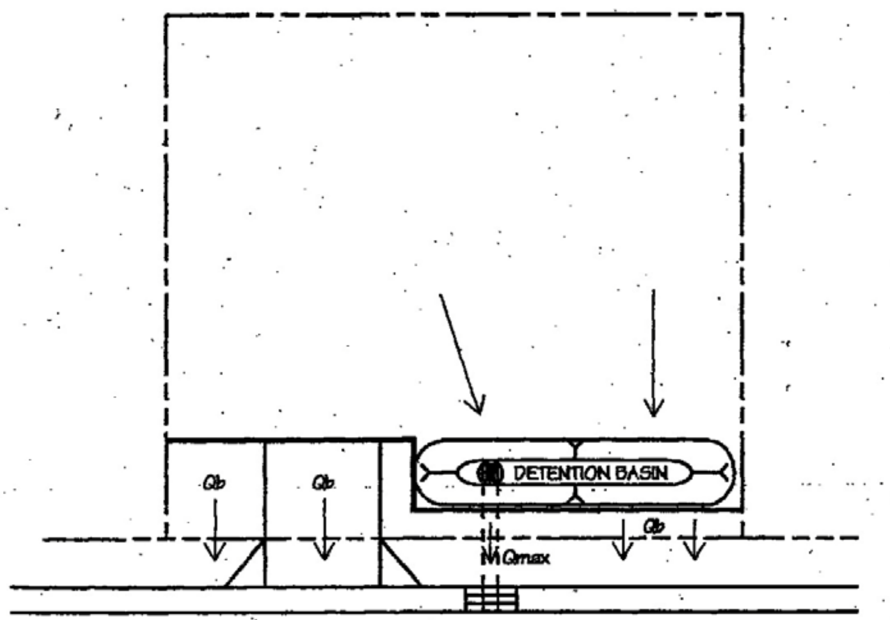
Figure 10-2
Calculating Storage Volume

BASIN SHAPE		
BASIN TYPE	VERTICAL WALLS AND/OR PRISMATIC BASINS	FAIRLY UNIFORM SHAPE AND SIDE SLOPES OR HIGHLY IRREGULAR SHAPE AND SIDE SLOPES
VOLUME CALCULATION METHOD	AVERAGE END AREA METHOD	CONIC METHOD
EQUATION	$V = \left(\frac{A_n + A_{n+1}}{2}\right)L$	$V = \Sigma V_{n \text{ to } n+1}$ $V_{n \text{ to } n+1} = [A_n + A_{n+1} + (A_n A_{n+1})^{5}]\frac{h}{3}$

WHERE: V = VOLUME (ft)
A_n = HORIZONTAL AREA (ft²) AT ELEVATION "n"
A_{n+1} = HORIZONTAL AREA (ft²) AT ELEVATION "n+1"
h = VERTICAL HEIGHT (ft) BETWEEN ELEVATION "n" AND "n+1"
$V_{n \text{ to } n+1}$ = VOLUME BETWEEN ELEVATION "n" AND "n+1"
L = LENGTH (ft) BETWEEN TWO ENDS

NOTE: THE ABOVE EQUATIONS MAY BE USED IN SUCCESSION FOR INCREMENTAL HEIGHTS WITHIN A BASIN. AN AREA SHOULD BE SELECTED AT ALL SIGNIFICANT CHANGES IN SHAPE OR SIDE SLOPE.

Figure 10-3
Total Site Runoff



Maximum release from detention pond $Qmax \leq$ Historic peak Q_h minus direct bypass runoff Qb
$Qmax \leq Q_h - Qb$

10.3.3 Hydrologic Design Methods and Criteria

Hydrologic design of detention facilities is based on the type of facility (regional vs. local) and the method used to estimate the runoff (UH Method vs. Simplified Procedures). If the UH Method is used, a full hydrograph is available for traditional storage routing. If a Simplified Procedure (Rational Method or FAA Method) is used, only a peak flow rate and volume are available. The following sections discuss the procedures for these two methods. Procedures used to calculate storm runoff are described in Section 3, Runoff Analysis, of these Drainage Criteria.

UH Method

The UH Method may be used to develop inflow hydrographs for hydrologic basins of any size, and may be used for local and regional facilities. Inflow hydrographs shall be based on ultimate development conditions.

This method can calculate a hydrograph for any location in the hydrologic basin. The data input file must be structured so that the proposed detention basin site is a hydrograph-routing or hydrograph-combining point.

The controlled outlet capacity has direct influence on the required size of the basin. The outflow limitation can be based on either the existing undeveloped peak flow from the hydrologic basin or on limitations in the capacity of the downstream conveyance system (based on a hydrologic analysis of local conditions), whichever is more restrictive. The design maximum outlet capacity of a regional facility must be coordinated with the City.

After the inflow hydrograph has been calculated and the outflow limits have been determined, the required storage volume can be estimated. Separate methods for calculating required storage are used depending on the method used to estimate the inflow hydrograph. In order to calculate the required storage volume at a particular detention basin site, the following information must be prepared:

1. Inflow hydrograph
2. Outlet capacity limitation
3. Proposed outlet discharge vs. elevation data for the proposed basin site
4. Proposed storage vs. elevation data for the proposed basin site
5. Proposed drain time for the proposed basin site

Computer programs can be used to determine the required storage volume and outflow limitation based on a reservoir routing procedure. It is an iterative process wherein the designer typically makes an initial estimate of outlet size then runs the program. The output is reviewed and changes made to the outlet configuration as needed until the desired degree of peak flood attenuation and acceptable drain time is achieved.

The modified Puls methodology for reservoir routing is recommended. The following procedure is presented in Hydrology for Engineers (Linsley 1975).

The principle of mass continuity for a channel reach can be expressed by the equation:

$$(I-D)\Delta t = \Delta S \tag{10.1}$$

Where:

I = the inflow rate
D = the discharge rate
Δt = the time interval
ΔS = the change in storage

If the average rate of flow during a given time period is equal to the average of the flows at the beginning and end of the period, the equation can be expressed as follows:

$$(I_1 + I_2)\ \Delta t / 2 - (D_1 + D_2)\ \Delta t / 2 = S_2 - S_1 \qquad (10.2)$$

Where the subscripts 1 and 2 refer to the beginning and end of time period t. Rearranging the equation gives the following form used for the Modified Puls method:

$$I_1 + I_2 + (2S_1 / \Delta t - D_1) = (2S_2 / \Delta t + D_2) \qquad (10.3)$$

FAA Method

The minimum required volume can also be determined using the FAA Method for hydrologic basins with a total area of less than 160 acres. The FAA Method was developed by the FAA in 1966 and modified by Guo (1999b).

The input required for the FAA Method includes:

A = Area of the catchment tributary to the storage facility (ac)
C = Runoff coefficient
Q_p = Pre-developed flow rate (allowable release rate from the detention facility) (cfs)
T_c = Time of concentration for the catchment (min)
P_1 = 1-hour design rainfall depth (in)

The calculations are best set up in a tabular (spreadsheet) form with each 5-minute increment in duration entered in rows and the following variables entered, or calculated, in columns:

1. Storm duration time, T (min), up to 180 minutes.
2. Rainfall intensity, I (in/hr), determined based on T_c and the values in Section 2, Rainfall.
3. Inflow volume, V_i (ft^3), calculated as the cumulative volume at each storm duration:

   $$V_i = CIA\ (60T) \qquad (10.4)$$

4. Outflow adjustment factor, m (Guo 1999a):

   $$m = ½\ (1 + T_c/T) \text{ for } 0.5 \leq m \leq 1 \text{ and } T \geq T_c \qquad (10.5)$$

5. The calculated average outflow rate, Q_{av} (cfs) over the duration T:

   $$Q_{av} = mQ_{po} \qquad (10.6)$$

6. The calculated outflow volume, V_o (ft^3), during the given duration and the adjustment factor at that duration calculated using:

   $$V_o = Q_{av}\ (60T) \qquad (10.7)$$

7. The required storage volume, V_s (ft^3):

$$V_s = V_i - V_o \tag{10.8}$$

The value of V_s increases with time, reaches a maximum value, and then starts to decrease. The maximum value of V_s is the required storage volume for the detention facility. Sample calculations using this procedure are presented in the Design Example in Section 10.9. The modified FAA Worksheet performs these calculations.

Compensating Detention Analysis

If any storm runoff will be discharged from the site without first being routed through a detention pond, on-site detention facilities are to be designed using the "compensating detention procedure". The total of all undetained areas shall not exceed 5% of the development area. Compensating detention is based on the following assumptions:

1. The minor and major design storm peak discharge from the property from detained and undetained area, when added together, will be no greater than the allowable discharge. Therefore, the more undetained release of storm runoff from the site, the less the detention pond is permitted to release, which requires proportionally larger detention volume.
2. Regardless of the method used, the volume of the detention pond must be adjusted to result in reduced discharge rates. In the UH Method, the detention volume is determined using actual runoff hydrographs and storage routing based on the outlet configuration. For the Rational and FAA Methods, the increase in volume is accomplished simply by computing the volume based on the *entire* property area, not just the area tributary to the detention pond. This is a reasonable assumption given the 5% area size limitation on undetained area.

The compensating detention procedure is given in the following six steps. This procedure applies to the UH, Rational, and FAA Methods for determining detention volumes, except as specifically noted otherwise.

1. If the undetained area is less than 5% of the total project area or 5,000-square feet, whichever is less, continue. If not, then the undetained area must either be reduced in size or the site layout revised to be in compliance.
2. Determine the allowable release rate for the minor and major design storms based on the pre-project site conditions using the *entire* site area. See Table 10-1.
3. Determine the post-project peak runoff rates for the minor and major design storms for the undetained area only.
4. Determine the *adjusted* allowable release rates by subtracting the runoff rates from the post-project, undetained area from the allowable release rates in Step 2.
5. Determine minimum required minor and major design storm storage volumes for the area tributary to the detention pond using the adjusted allowable release rates. If using the Rational or FAA Methods, the storage volume is determined using the equations in this section based on the *entire* project area, not just the area tributary to the detention pond.

6. Determine the final outlet configuration that results in the *adjusted* allowable release rates at the computed detention volumes for the entire site.

Over-Detention Analysis

Over-detention is detaining developed conditions peak flows to release at rates that are lower than pre-developed conditions. Over-detention is sometimes required to meet capacity limitations of downstream drainage facilities. Specific over-detention requirements may be identified in the City's Stormwater Master Plan for the watershed within which the development is proposed.

Over-detention requirements are unique for each basin or watershed and are often based on a detailed hydrologic investigation using hydrograph methods to generate peak flows and route the flows through the drainage system. Simplified detention volume and release rate methods are not appropriate, and over-detention volume and release rate requirements shall be determined using the UH Method.

10.3.4 Outlet Design

Outlet structures from detention ponds are typically pipes with control structures and/or overflow weirs. The hydraulic design procedure for sizing overflow weirs is as follows:

Weir Flow

The general form of the equation for horizontal broad-crested weirs is:

$$Q = CL(H)^{3/2} \tag{10.9}$$

Where:

Q = Discharge (cfs)
C = Weir coefficient (see Figure 10-4)
L = Horizontal length (ft)
H = Total energy head (ft)

Another common weir is the v-notch, the equation for which is:

$$Q = 2.5 \tan(\Theta/2)H^{5/2} \tag{10.10}$$

Where:

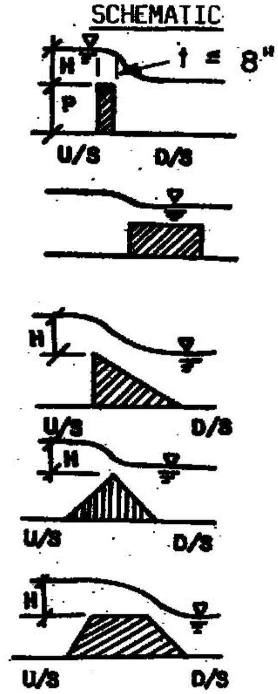
Θ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow, effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater to the headwater depth. The example calculation for a weir design shown in Figure 10-5 illustrates the submergence check.

Figure 10-4
Weir Flow Coefficients (Design of Small Dams, USBR 1977)

SHAPE	COEFFICIENT	COMMENTS
Sharp Crested	-	
Projection Ratio (H/P = 0.4)	3.4	H< 1.0
Projection Ratio (H/P = 2.0)	4.0	H> 1.0
Broad Crested	-	
W/Sharp U/S Corner	2.6	Minimum Value
W/Rounded U/S Corner	3.1	Critical Depth
Triangular Section	-	
A) Vertical U/S Slope	-	
1:1 D/S Slope	3.8	H > 0.7
4:1 D/S Slope	3.2	H> 0.7
10:1 D/S Slope	2.9	H> 0.7
B) 1:1 U/S Slope	-	
1:1 D/S Slope	3.8	H >0.5
3:1 D/S Slope	3.5	H >0.5
Trapezoidal Section		
1:1 U/S Slope, 2:1 D/S Slope	3.4	H > 1.0
2:1 U/S Slope, 2:1 D/S Slope	3.4	H > 1.0
Road Crossings		
Gravel	3.0	H ≥ 1.0
Paved	3.1	H ≥ 1.0

SCHEMATIC



ADJUSTMENTS FOR TAILWATER

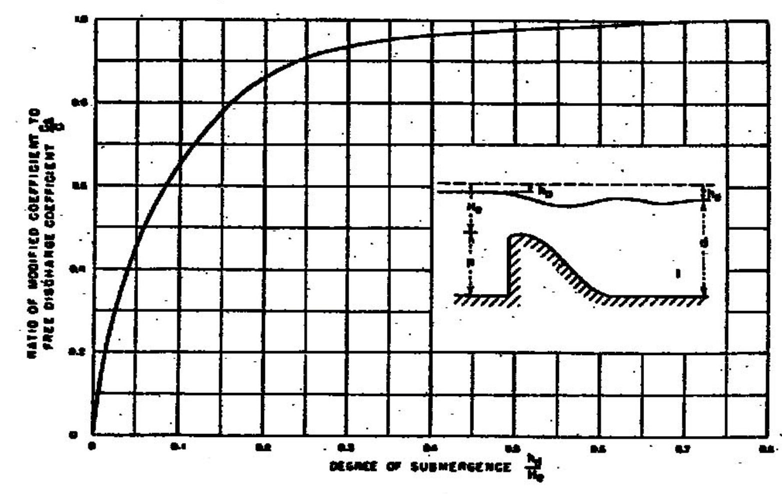
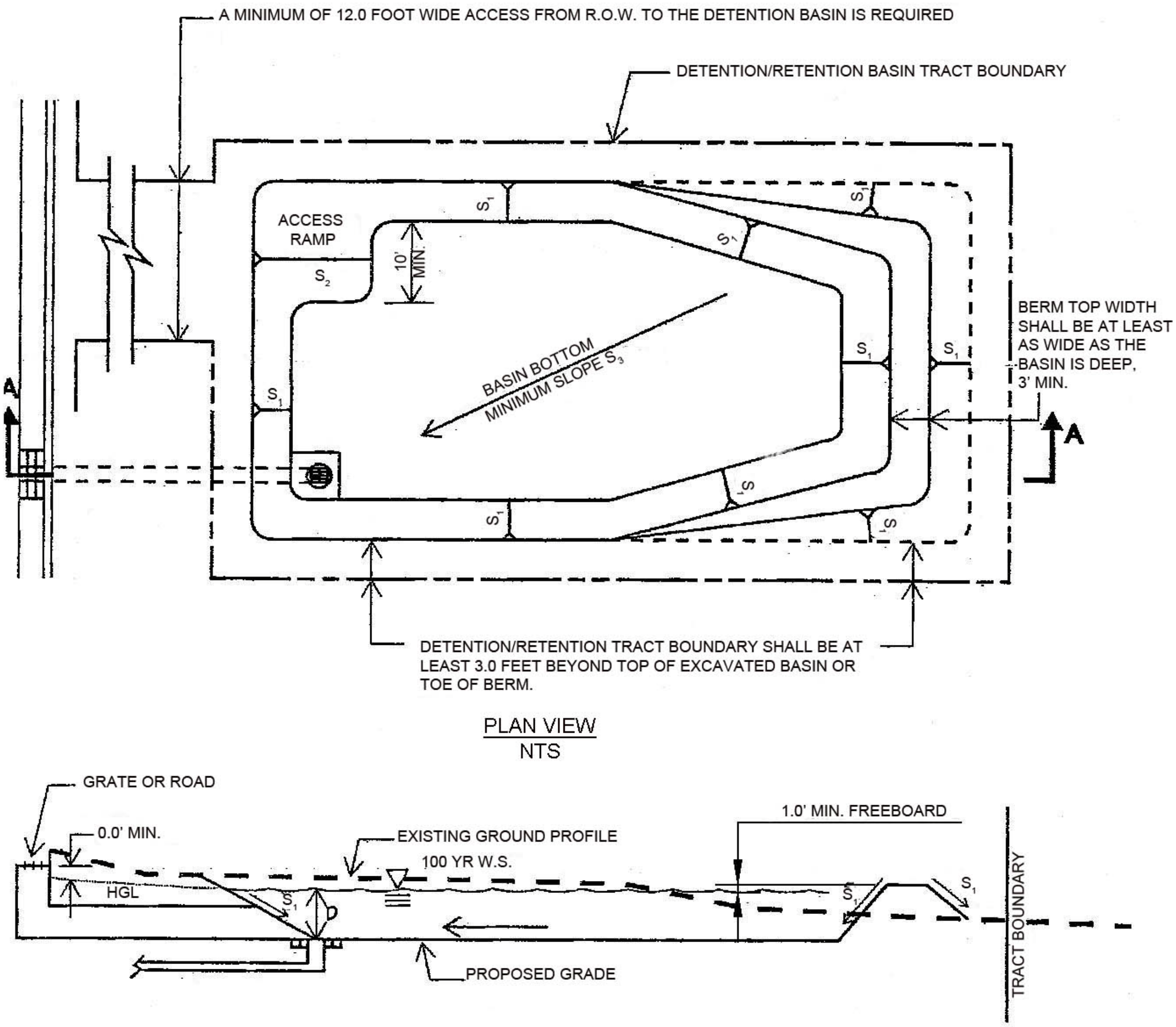


Figure 10-5
Detention/Retention Basin Geometric Requirements



NOTES:

S_1 MAXIMUM:	4H:1V
S_2 MAXIMUM:	10H:1V FOR ACCESS RAMP, ALL SURFACES
S_3 MINIMUM:	0.5% FOR CONCRETE CHANNEL 1.0% FOR ASPHALT (PARKING LOT) 2.0% FOR ALL OTHER SURFACES
D MAXIMUM:	4' RETENTION BASIN 8' WET OR DRY DETENTION FACILITY >8' SPECIAL APPROVAL REQUIRED, BUT MAY BE ALLOWED FOR MULTIPLE USE PONDS OR FOR STEEP TERRAINS
D MINIMUM:	4' WET PONDS

Orifice Flow

The equation governing the hydraulic design of an orifice opening and plate is:

$$Q = C_d A\,(2gh)^{1/2} \tag{10.11}$$

Where:

Q = Flow (cfs)
C_d = Orifice coefficient
A = Area (ft^2)
g = Gravitational acceleration (32.2 ft/s^2)
h = Head on orifice measured from center of the opening (ft)

An orifice coefficient (C_d) value of 0.65 shall be used for sizing of squared-edged orifice openings and plates on outlet structures.

10.3.5 Off-Site Flows

There are two methods by which off-site flows can be routed relative to the new development. These are routing around the new development and routing through the new development.

Routing of Off-Site Storm Runoff

1. **Routing Around the New Development:** Ideally, new development will be graded so off-site storm runoff is routed separately through or around the development. If this method is chosen, off-site flows must be routed to their historic path immediately downstream of the new development and not impact adjacent properties. Concentrated flows draining to the development should be maintained as concentrated flows downstream of the development, and sheet flows should be maintained as sheet flow.
2. **Routing Through the New Development:** If the site cannot be graded to route off-site flows away from the detention pond, the development's detention pond must be designed to account for any off-site flow that comes into the new development and into the new detention pond. It is possible that routing off-site runoff through the new detention pond will significantly alter the volume and outlet requirements of the new detention pond.

The effects of routing storm runoff from off-site areas through the new detention pond shall be determined by UH Method. A hydrograph of the off-site area, assuming ultimate development conditions without detention, shall be generated to achieve a total volume of runoff. The developed condition hydrograph shall then be truncated at the allowable release rate and extended in time such that the total runoff volume is the same. This "modified" hydrograph is a reasonable representation of the ultimate development runoff that is routed through a detention pond within the offsite development.

The modified off-site hydrograph is then added to the developed conditions hydrograph for the new development (assuming no on-site detention). This composite hydrograph shall then be routed through the new detention pond to verify that the release rates from the new detention pond meet allowable release rates. In this instance, the allowable release rate is the sum of the allowable release rates from *both* the off-site area and the new development. The design of the detention pond may require iteration on the detention volume and outlet configuration to achieve the required results.

10.4 DESIGN STANDARDS FOR OPEN SPACE DETENTION

10.4.1 Grading Requirements

For proper function and safety considerations, geometric requirements shall be as shown on Figure 10-5, and conform to the following criteria:

1. Pond bottoms shall be designed with a 0.5% minimum longitudinal and a 2.0% minimum cross slope.
2. Pond side slopes shall be 4(H):1(V), maximum. Side slopes steeper than 4(H):1(V) must be approved by the City and, if approved, protected from erosion and instability and not require more than routine maintenance.
3. Detention storage in parking lots shall have a maximum depth of 12 inches during the Major storm.
4. Appropriate measures (typically an all-weather access road to the basin) shall be included to allow for access by maintenance equipment. Maintenance equipment must be able to reach the bottom of the facility safely and have adequate space to operate.
5. Check finished floor elevation of any structure near the detention basin. The 100-year water surface should be 1 foot below the finished floor.

Use of Retaining Walls

Retaining walls within detention basins are generally discouraged; however, if walls are unavoidable, low-height walls less than 30 inches that are constructed of natural rock or landscape block are preferred. Design must address long-term maintenance access, safety, stability, seepage, structural soundness and aesthetics.

If several retaining walls are used in a terracing configuration, horizontal separation of at least 4 feet shall be provided. Any future outfalls to the basin shall be designed and constructed concurrently with the detention basin. This eliminates future disturbance of the retaining walls, which may jeopardize the wall's structural integrity, in order to construct the future outfall. Foundation walls of buildings shall not be used as detention basin retaining walls.

If accepted by the City, any retaining walls exceeding a height of 30 inches (as measured from the ground line at the base to the top of the wall) shall be provided with guards and shall require a Building Permit. All guards shall be designed to meet International Building Code (IBC) requirements.

Walled-in or steep-sided basins should be located away from major pedestrian routes, and emergency egress routes should be provided. Site lighting may also be required by the City to discourage illicit activity in walled-in basins.

A professional engineer licensed by the state of Wyoming shall perform a structural design of the retaining walls. The wall design and calculations shall be stamped by the professional engineer and submitted to the City. The structural design requirements and details for the retaining walls shall be included in the construction drawings.

10.4.2 Freeboard Requirements

Open space detention/retention facilities shall be designed with a minimum freeboard of 1 foot above the emergency spillway water surface elevation.

10.4.3 Trickle Flow Channels

Most drainage conveyance systems are designed to divert even minor nuisance flows to stormwater storage facilities. For dry basins, this can present an aesthetic and maintenance problem. A trickle channel shall be provided to convey low flows to the outlet structure. See Figures 10-6 and 10-7.

Ponds shall have a trickle channel with a capacity of 1% to 3% of the peak design storm inflow to the pond and a minimum slope of 0.5%. The outlet facility for a retention basin shall be a dry well or riprap-filled dissipation pit. For a detention basin, nuisance flows shall be conveyed to the basin outlet. Low-flow and trickle channels shall be included in the channel cross-section to improve channel conditions during sustained flow conditions and frequent storm events.

10.4.4 Inlet and Outlet Configuration

Inlets to the detention/retention pond can be surface rundowns or by storm drain system. All points of inflow to the pond shall be protected to prevent erosion. The design of protection measures shall be based on no water in the pond.

Outlet structures shall control discharges to approved release rates. The minimum diameter for outlet pipes shall be 12 inches. Multiple pipe outlets may be required to control different design frequencies. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pond or at the top of the minimum pool, if present. The outlet pipe(s) shall discharge into a standard manhole or into a drainageway with proper erosion protection for the proposed structure. If an orifice plate is required to control the release rates, the plate(s) shall be fastened to a structure, such as a modified inlet or headwall, to facilitate back flushing of the discharge pipe(s).

An emergency spillway shall be designed to pass the peak discharge safely for the Major storm event, assuming that the outlet structure is ineffective and the upstream basin is fully developed to current zoning or current development levels, whichever produces the greatest runoff. The flow from offsite areas specified in section 10.3.5 shall be added to this amount. Flows from the emergency spillway shall be routed safely to the site downstream discharge point. See Sections 4 and 9 for design of the spillway outlet.

10.4.5 Embankment Protection

Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a larger than 100-year storm occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. Structures shall not be permitted in the path of the emergency spillway or overflow.

Figure 10-6
Dry Detention Basin Trickle Flow Conveyance

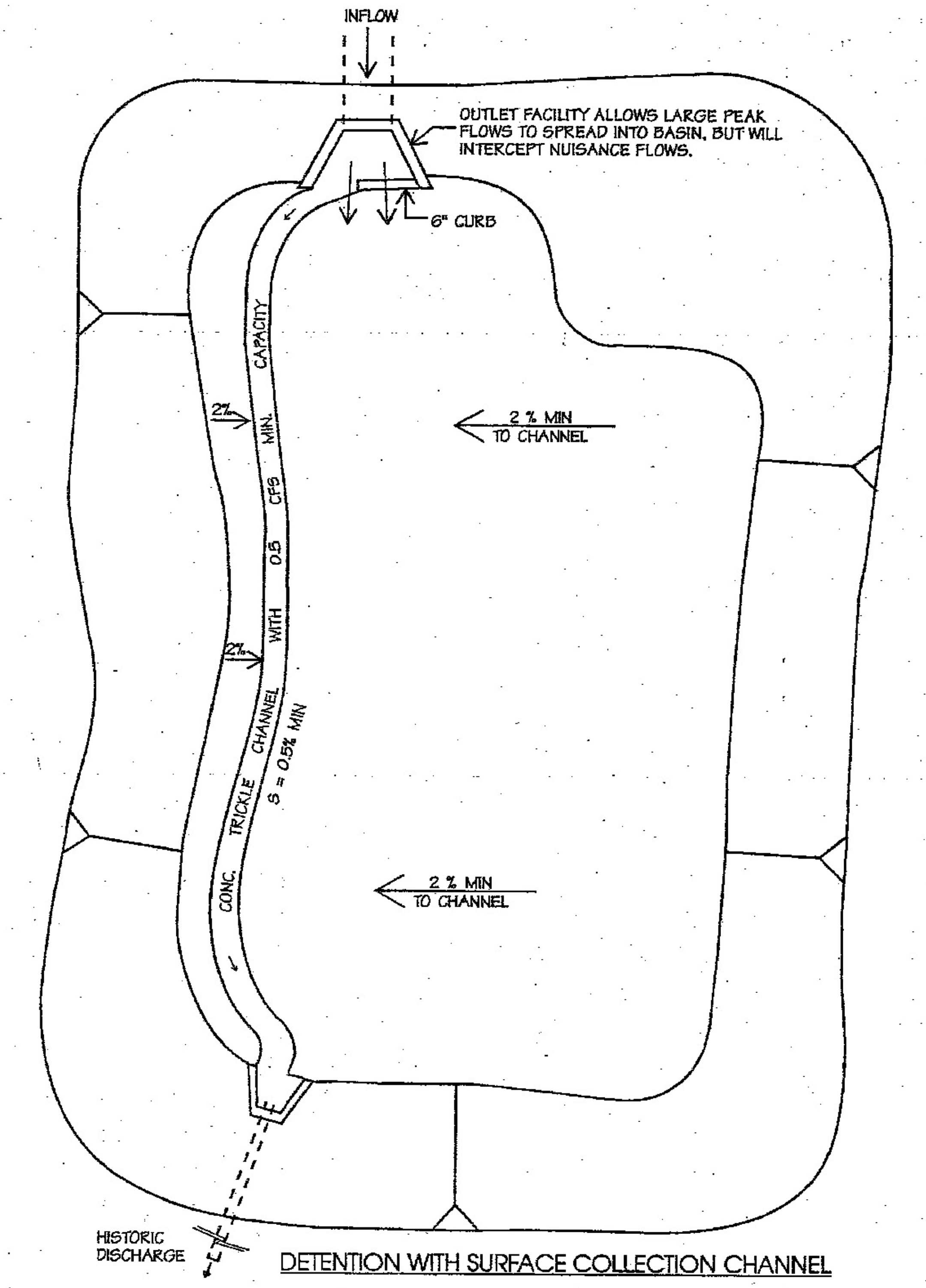
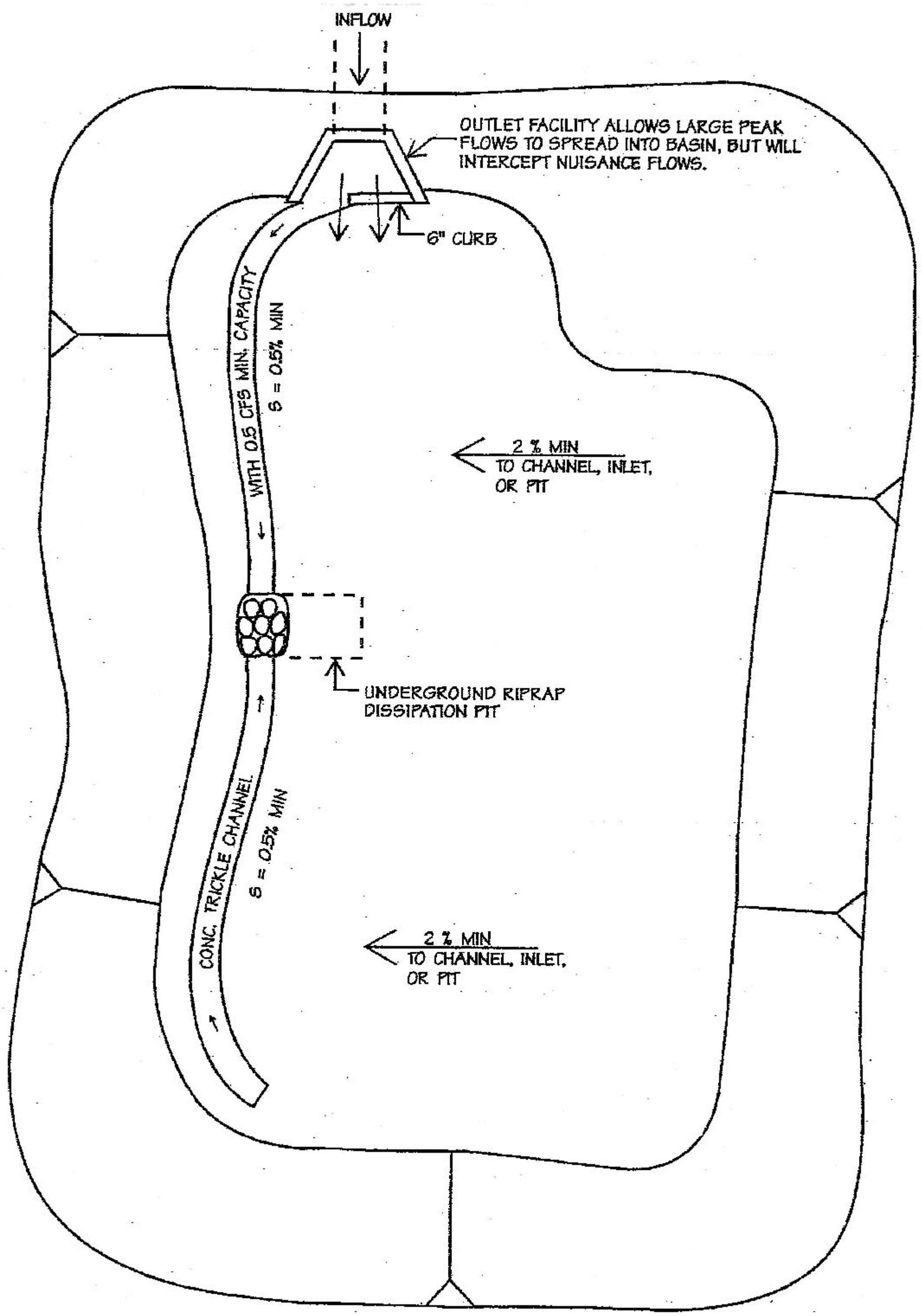


Figure 10-7
Dry Retention Basin Trickle Flow Conveyance



RETENTION WITH SURFACE COLLECTION CHANNEL
(THE SAME CONCEPT APPLIES TO AN UNDERGROUND SYSTEM)

10.4.6 Vegetation Requirements

After final grading, the slopes and bottom of each detention and retention basin shall be protected from erosion by seeding and mulching, sodding, or other approved groundcover and shall be in accordance with City specifications.

The planting of trees and shrubs on the non-embankment slopes of stormwater basins is also encouraged. Temporary and/or permanent irrigation systems shall be provided as required for the type of groundcover and landscape installed and approved.

10.4.7 Maintenance Access

Detention pond right-of-way shall be as required to contain the required storage volume, including freeboard, outlets and embankments, and maintenance access around the entire perimeter. All detention ponds to be maintained by the City shall be provided with maintenance access by adjacent public right-of-way or easements. Local, minor detention ponds will normally remain in private ownership unless upstream basins from outside of the subject property contribute to the pond or runoff from more than one property contributes to the pond. Maintenance access to detention ponds shall be provided to the pond bottom and have a maximum slope of 10%.

10.4.8 Retention

When allowed, retention will be designed with the following minimum requirements:

1. Retention basins must drain within 48 hours *of all storm events up to* the 100-year storm event, beginning either at the start of the storm or when runoff first reaches the basin.
2. Total runoff volume shall be determined by calculating imperviousness, then converting to this to a curve number (CN) following the procedures in Section 3, Runoff Analysis. Using the CN, excess precipitation for the design storm is determined using procedures described in Section 3.4.3, Precipitation Losses. The design storm shall be the 24-hour precipitation amount of 4 inches multiplied by a factor of 1.5.
3. An emergency overflow from the retention area shall be provided with a minimum capacity of the 100-year peak inflow rate. Flows from the emergency spillway shall be routed safely to the site downstream discharge point. See sections 4 and 9 for design of the spillway outlet. The invert elevation of the overflow shall be located at the maximum retention volume water surface.
4. A minimum of 1 foot of freeboard shall be provided above the maximum retention volume water surface elevation to the crest of the retention basin embankment.

The design of the retention basin must be supported by vertical hydraulic infiltration data for subsurface soil and/or rock, obtained from an appropriate geotechnical engineering test. Test data shall then be used to determine the required size of the retention basin and the time required for complete infiltration of the 100-year storm event. One of the following models shall be used to demonstrate the design meets all criteria:

- ModFLOW-2005, Ver. 1.8 (USGS 2009)
- SEEP/W (Geo-Slope International, Ltd. 2007)

- Equivalent model (upon City approval)

When using vertical hydraulic conductivity data in these calculations, a saturated K-value shall be used only when it has been demonstrated that the infiltration media over the entire length of the infiltration path are saturated. If the infiltration path is characterized by unsaturated conditions, a partially saturated K-value shall be used for infiltration calculations.

Measures that minimize sediment from entering and clogging the pond surface shall be implemented. These include pre-sediment basins and/or frequent sediment removal. Depth of storage shall be minimized and surface area shall be maximized for infiltration purposes.

An operation maintenance plan that includes monitoring and reporting requirements and penalties for non-compliance shall be developed and be part of the development agreement. This plan may include pumping to meet the drain time requirements and should consider that there may be power outages during a major storm.

10.4.9 State Engineer's Office

Permits from the State Engineer's Office are required for any dams (including detention facilities) with a storage volume of 50 acre-ft or greater or a height of 20 feet or greater.

The Special Application filing procedures apply to flood control reservoirs and dams between 6 feet and 20 feet in height and with an inactive capacity between 20 acre-feet and 50 acre-feet if there is an 18-inch diameter, or larger, uncontrolled outlet pipe.

Dam or detention facilities are exempt from the State Engineer permitting process when they:

1. Are pit-type or excavated flood control impoundments that are designed with an outlet device that will evacuate the impoundment within 24 hours.
2. Have a storage capacity of 50 acre-feet or less and a height of 6 feet or less as measured from the downstream toe to the crest of the dam. Under this exemption the outlet pipe must be a minimum of 18 inches in diameter, be uncontrolled, and totally evacuate the impoundment.

State regulations (WYSEO 2006) may be updated or revised periodically. It is the responsibility of the owner and the designer to comply with all state requirements for design and permitting.

10.5 DESIGN STANDARDS FOR PARKING LOT DETENTION

Stormwater detention can be provided in parking lots on commercial sites according to the following criteria:

1. The maximum allowable design depth of ponding in parking lots for the 100-year flood is 12 inches.
2. Where a drop inlet is used to discharge to a storm drain or drainageway, the minimum pipe size for the outlet is 8-inch diameter, provided that it can convey 120 percent of the 100-year outflow. Where a weir and a small diameter outlet through a curb are used, the size and shape are dependent on the discharge/storage requirements. A minimum 4-inch-diameter pipe size is recommended.

3. To assure that the detention facility performs as designed, maintenance access shall be provided.
4. The outlet shall be designed to minimize unauthorized modifications, which affect function. Any repaving of the parking lot shall be evaluated for impact on volume and release rates and is subject to approval by the City.
5. All parking lot detention areas shall have multiple signs posted identifying the detention basin area. The signs shall have a minimum area of 1.5 square feet and contain the following message: **WARNING: This area is a detention basin and is subject to periodic flooding to a depth of (provide design depth).** Any suitable materials and geometry of the sign are permissible, subject to approval by the City.

10.6 DESIGN STANDARDS FOR UNDERGROUND DETENTION

Underground detention is discouraged for use in the City for the following reasons:

- Underground detention is not visible; therefore, it tends to be "out-of-sight, out-of-mind." As a result, these devices do not typically receive regular maintenance, nor is their performance monitored.
- Maintenance access is often poor, which is a deterrent to maintenance.
- Anaerobic (absence of dissolved oxygen) conditions in bottom sediments are more likely to develop in underground devices. This condition can release pollutants that were bound to the sediment and cause bad odors.

Nevertheless, there are cases where the use of such facilities is necessary due to space constraints in smaller, urban redevelopment sites. The City will consider the use of underground detention under these circumstances; however, the applicant must comply with the following restrictions prior to receiving authorization for its use:

- Clear evidence must be provided documenting why detention cannot be provided on the ground surface and why the use of an underground facility is the best choice for the site, considering factors such as initial installation, maintenance, and ability to assure long-term function.
- The Water Quality Capture Volume (WQCV, see Section 12) should still be provided aboveground, even if detention is provided below ground.

When no other alternative is practicable, the requirements for underground detention are as follows:

Materials

Underground detention shall be constructed using corrugated aluminum pipe (CAP), reinforced concrete pipe (RCP), concrete vaults, or approved equivalents. Galvanized or aluminized pipes are not acceptable. The pipe thickness, cover, bedding, and backfill shall be designed to withstand HS-20 loading, or as otherwise required by the City.

Configuration

Pipe or vault segments shall be sufficient in number, height, and length to provide the required minimum storage volume. The minimum headroom height of the pipe or vault segments shall be 48 inches to permit maintenance. If parallel pipes are used, the pipe segments shall be placed side by side and connected at both ends by elbow and tee fittings. The pipe segments shall be continuously sloped at a minimum of 0.25 percent to the outlet. Manholes for maintenance access shall be placed in the tee fittings, bends, and in the straight segments of the pipe, when required.

Permanent buildings or structures shall not be placed directly above the underground detention.

Inlet and Outlet Design

Inlets to underground detention facilities can be surface inlets, pipes, and/or a local private storm drain system. Outlets from underground detention shall consist of a short (maximum 50-foot) length(s) of pipe with an 8-inch minimum diameter that can convey 120 percent of the 100-year outflow. A two-pipe outlet may be required to control both minor and major design return periods. The invert of the lowest outlet pipe shall be set at the lowest point in the detention vault. The outlet pipe(s) shall discharge into a standard manhole or standard inlet or into an open drainageway with erosion protection. If an orifice plate is required to control the release rates, the plate shall have a hinge on one side to open into the detention pipes to facilitate back flushing of the outlet pipe(s) and be firmly bolted or secured to the wall to prevent leakage around the edges.

Maintenance Access

Access for maintenance of the detention facility shall be provided. Maintenance access designs shall take into consideration Occupational Safety and Health Administration (OSHA) requirements for confined space entry. Ensure that failure of underground detention is clearly evident from above ground through the use of observation ports and wells.

Maintenance Covenant

The City shall require a maintenance covenant from the Owner, which shall run with the property.

10.7 WATER QUALITY ENHANCEMENT

The outlet for a detention basin can be designed to allow for a slower rate of release and provide time for particulate pollutants to settle out of the stormwater. Such basins are called extended detention basins (EDB). It is recommended that an EDB outlet be designed to drain a full basin completely in a minimum of 40 hours, which provides removal of a significant portion of insoluble pollutants with proper operation and maintenance. A retention pond (RP) is a sedimentation facility designed to have a permanent water pool. This pool of water is mixed with stormwater during storm runoff events from frequently occurring storms and allows for sedimentation to occur, enhancing water quality.

For frequently occurring storms, both EDBs and RPs capture the total runoff as a surcharge. However, in the case of RPs, the stormwater is allowed to mix with the permanent pool water as it rises above the permanent pool level. All surcharged water above the permanent pool level is to be released over 40 hours. Details related to the design of EDBs and RPs are discussed in Section 12, *Stormwater Quality Enhancement* of these Drainage Criteria.

For the 100-year detention, the water quality volume is considered to be a part of the volume required for detention purposes.

10.8 MAINTENANCE

Continued maintenance of detention facilities is necessary to ensure they will function as designed. Maintenance of detention facilities includes mowing, removal of debris and sediment, and restoring damaged structures.

All detention facilities will be designed to minimize and facilitate maintenance to preserve their function, and shall:

1. Include access to the entire detention facility by dedication of rights-of-way, easements, and tracts of land specifically for drainage purposes. Tracts or easement dedications shall prohibit uses and the construction of permanent improvements that restrict or block access. Specifically, detention and retention basins in subdivisions shall be located in tracts owned by the City or with an easement granted to the City. Basins for individual sites require neither a tract nor an easement but do require a maintenance agreement with the City.
2. Be maintained by the property owner, the developer and/or a homeowners association or the City, depending on the type of facility. Should the property owner fail to maintain drainage facilities adequately, the City reserves the right to enter the property, upon proper notice, for the purpose of performing drainage maintenance. All maintenance costs incurred by the City shall be assessed against the owner(s) of the property.

10.9 DESIGN EXAMPLE – DETENTION VOLUME

FAA Method Analysis

Use the FAA method to determine the required detention volume for the 100-year storm event for a 15-acre site that will have a developed percentage imperviousness of 45%. The NRCS soil survey shows the site has hydrologic soil group B soils. The allowable release rate from the basin has to be limited to the unit values in Table 10-1. The time of concentration has been calculated at 12 minutes. The 100-year, 1-hour point precipitation is 2.32 inches.

A runoff coefficient, *C*, of 0.51 is determined using methods in Section 3 Runoff Analysis. The calculations are shown in Table 10-1 in spreadsheet form using the UD-Detention workbook. This workbook is available for free at the UDFCD website at http://www.udfcd.org/downloads/down_software.htm.

Table 10-1
FAA Method Example Calculations
(from UD-Detention Workbook)

Rainfall Duration (minutes)	Rainfall Intensity (inches/hr)	Inflow Volume (cubic feet)	Adjustment Factor ("m")	Average Outflow (cfs)	Outflow Volume (cubic feet)	Storage Volume (cubic feet)
0	0.00	0	0.00	0.00	0	0
10	6.22	28,552	1.00	6.45	3,870	24,682
20	4.52	41,469	0.80	5.16	6,192	35,277
30	3.60	49,573	0.70	4.52	8,127	41,446
40	3.02	55,427	0.65	4.19	10,062	45,365
50	2.61	60,000	0.62	4.00	11,997	48,003
60	2.31	63,755	0.60	3.87	13,932	49,823
70	2.08	66,943	0.59	3.78	15,867	51,076
80	1.90	69,717	0.58	3.71	17,802	51,915
90	1.75	72,175	0.57	3.66	19,737	52,438
100	1.62	74,385	0.56	3.61	21,672	52,713
110	1.51	76,395	0.55	3.58	23,607	52,788
120	1.42	78,239	0.55	3.55	25,542	52,697
130	1.34	79,945	0.55	3.52	27,477	52,468
140	1.27	81,534	0.54	3.50	29,412	52,122
150	1.21	83,021	0.54	3.48	31,347	51,674
160	1.15	84,419	0.54	3.47	33,282	51,137
170	1.10	85,740	0.54	3.45	35,217	50,523
180	1.05	86,992	0.53	3.44	37,152	49,840
190	1.01	88,183	0.53	3.43	39,087	49,096
200	0.97	89,319	0.53	3.42	41,022	48,297
210	0.94	90,405	0.53	3.41	42,957	47,448
220	0.91	91,446	0.53	3.40	44,892	46,554
230	0.88	92,446	0.53	3.39	46,827	45,619
240	0.85	93,408	0.53	3.39	48,762	44,646
250	0.82	94,335	0.52	3.38	50,697	43,638
260	0.80	95,230	0.52	3.37	52,632	42,598
270	0.78	96,095	0.52	3.37	54,567	41,528
280	0.75	96,933	0.52	3.36	56,502	40,431
290	0.73	97,745	0.52	3.36	58,437	39,308
300	0.72	98,533	0.52	3.35	60,372	38,161
310	0.70	99,299	0.52	3.35	62,307	36,992
320	0.68	100,043	0.52	3.35	64,242	35,801
330	0.67	100,768	0.52	3.34	66,177	34,591
340	0.65	101,474	0.52	3.34	68,112	33,362

Table 10-1
FAA Method Example Calculations
(from UD-Detention Workbook)

Rainfall Duration (minutes)	Rainfall Intensity (inches/hr)	Inflow Volume (cubic feet)	Adjustment Factor ("m")	Average Outflow (cfs)	Outflow Volume (cubic feet)	Storage Volume (cubic feet)
350	0.64	102,162	0.52	3.34	70,047	32,115
360	0.62	102,834	0.52	3.33	71,982	30,852
370	0.61	103,490	0.52	3.33	73,917	29,573
380	0.60	104,131	0.52	3.33	75,852	28,279
390	0.59	104,757	0.52	3.32	77,787	26,970
400	0.57	105,371	0.52	3.32	79,722	25,649
410	0.56	105,971	0.51	3.32	81,657	24,314
420	0.55	106,559	0.51	3.32	83,592	22,967
430	0.54	107,135	0.51	3.32	85,527	21,608
440	0.53	107,699	0.51	3.31	87,462	20,237
450	0.52	108,254	0.51	3.31	89,397	18,857
460	0.52	108,797	0.51	3.31	91,332	17,465
470	0.51	109,331	0.51	3.31	93,267	16,064
480	0.50	109,856	0.51	3.31	95,202	14,654
490	0.49	110,371	0.51	3.30	97,137	13,234
500	0.48	110,877	0.51	3.30	99,072	11,805
510	0.48	111,375	0.51	3.30	101,007	10,368
520	0.47	111,865	0.51	3.30	102,942	8,923
530	0.46	112,347	0.51	3.30	104,877	7,470
540	0.46	112,822	0.51	3.30	106,812	6,010
550	0.45	113,289	0.51	3.30	108,747	4,542
560	0.44	113,749	0.51	3.29	110,682	3,067
570	0.44	114,203	0.51	3.29	112,617	1,586
580	0.43	114,649	0.51	3.29	114,552	97
590	0.42	115,090	0.51	3.29	116,487	-1,397
600	0.42	115,524	0.51	3.29	118,422	-2,898

Mod. FAA Major Storage Volume (cubic ft.) = 52,788

Mod. FAA Major Storage Volume (acre-ft.) = 1.2118

10.10 CHECKLIST

Several key considerations in the design of detention ponds that the designer must address include:

1. Grade earth slopes 4:1 or flatter.
2. Provide minimum freeboard of 1 foot.
3. Provide trickle channels in above-ground detention areas.
4. Protect embankment from overtopping conditions.
5. Provide outlet structures.
6. Provide signs as required.
7. Provide maintenance access.
8. Provide emergency spillway and check emergency overflow path as required.
9. Check finished floor elevation of any structure near the detention basin.
10. Ensure that failure of underground detention is clearly evident from above ground.

10.11 CITY ACCEPTANCE OF STORMWATER DETENTION/ RETENTION FACILITIES

The City and the Engineer shall make the final inspection of all drainage improvements in the project. If there are any items that are not in conformance with the City Specifications, the Private Developer and Engineer will be notified. The Private Developer shall be required to bring the items into conformance. On City contracted projects, the Contractor shall be notified and required to bring the items into conformance.

The City Engineering Division shall review the "Record Drawings." If the plans need to be revised or if additional information is required, a set of prints will be returned to the Engineer. The Engineer shall then revise and resubmit the "Record Drawings."

Private Development Projects shall provide a two-year warranty statement covering all the public improvements in the project.

When the public improvements have passed the final inspection, the "Record Drawings" have been stamped and approved, and the Warranty Statement has been provided, the City Engineer shall make a written recommendation to the Gillette City Council to accept the public improvements for maintenance.

The warranty period begins on the day that the City Council approves and accepts the public improvements.

During the warranty period, the Private Developer is responsible for repair work on any of the public improvements. The City Engineering Division will periodically inspect the public improvements and will notify the Private Developer if repair work is required. The Private Developer is responsible for having the repair work done prior to the end of the warranty period.

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SECTION ELEVEN
EROSION AND SEDIMENT CONTROL

11.1 PURPOSE

This section provides criteria for selection, design, installation and maintenance of sediment and erosion control Best Management Practices (BMPs) to reduce impacts to surface water resources from land disturbing construction activities. In addition, this section contains information regarding requirements for applying for coverage under City of Gillette Stormwater Discharge Permit for construction-related activities, and includes a set of erosion and sediment control BMP Fact Sheets for use and reference on construction projects.

11.2 INTRODUCTION AND REGULATORY BACKGROUND

Nationally, there is an increasing emphasis placed on protecting streams, lakes and reservoirs from receiving pollutant-laden stormwater discharges, whether from point or non-point sources. Through the National Pollutant Discharge Elimination (NPDES) Permitting system, specific regulations and policies have been developed to ensure the nation's surface waters are protected and include, but not limited to:

- NPDES program becoming more stringent with respect to numerical guidelines for non-point source pollution
- Identification of impaired waters and Total Maximum Daily Loads (TMDLs)
- Endangered Species Act (ESA)
- Federal, state and local stormwater and surface water quality programs

The City of Gillette is committed to meeting the goals and objectives of these regulations and has, therefore, developed specific criteria presented in this section for temporary, construction-related activities.

Following is a brief summary on the background of the stormwater regulations and how they apply to construction-related activities in Wyoming and the City of Gillette.

11.2.1 Federal Stormwater Program and Permits

In 1987 the United States Congress amended the federal Clean Water Act (CWA) to include the regulation of some sources of stormwater. The US Environmental Protection Agency (EPA) published regulations governing stormwater discharges in 1990. The NPDES Program, as part of the CWA, regulates stormwater discharges from three potential sources: municipal separate storm sewer systems (MS4s), construction activities, and industrial activities. Most stormwater discharges are considered point sources, and operators of these sources may be required to receive an NPDES permit before they can discharge. This permitting mechanism is designed to prevent stormwater runoff from washing harmful pollutants into local surface waters such as streams, rivers or lakes.

The Federal NPDES Program covers the following types of stormwater discharges:

- MS4s – Operators of large, medium and regulated small MS4s may be required to obtain authorization to discharge stormwater.
- Construction Activities – Operators of construction sites that are one acre or larger (including smaller sites that are part of a larger common plan of development) may be required to obtain authorization to discharge stormwater under an NPDES construction stormwater permit.
- Industrial Sites – Industrial activities may require authorization under an NPDES industrial stormwater permit for stormwater discharges.

11.2.2 Wyoming Stormwater Program and Permits

The WYDEQ was created by the Wyoming Legislature in 1973 after passage of the Environmental Quality Act. The WYDEQ is responsible for enforcing state and federal environmental laws including, but not limited to the:

- Clean Water Act
- National Pollutant Discharge Elimination System
- Environmental Quality Act

In 1991, the EPA granted the WYDEQ primacy for the stormwater program in Wyoming. Hence, the WYDEQ is responsible for administering and issuing two general permits, one specific for certain industrial activities defined in the federal regulations and the second for construction activities that disturb one or more acres. These permits are part of the WPDES Stormwater Program in Wyoming.

The WYDEQ's Water Quality Division has programs in place to ensure that these regulations are enforced and implemented in the state of Wyoming, including the following as they apply to stormwater quality enhancement:

- Watershed Program – addressing non-point sources and TMDLs;
- Water and Wastewater Program – addressing water and wastewater construction permitting; and
- WYPDES (Point Source) Program – which regulates discharges of pollutants that have the potential to reach surface waters of the state, including permits for stormwater discharges from regulated activities.

The WYPDES Stormwater Program requires permits for:

- Large construction – surface disturbance of 5 acres or more.
- Small construction – disturbance of at least 1 acre, but less than 5 acres.
- Non-stormwater activities for construction.
- Industrial sites.
- MS4s (currently applies to urbanized areas with population of 50,000 or more, or areas with impaired water bodies).

The WYDEQ's General Permits to Discharge Stormwater under the WYPDES Stormwater Program can be seen at the Wyoming DEQ website.

Part 8 of the WYDEQ's General Permits to Discharge Stormwater associated with Small or Large Construction Activities addresses the following general criteria for construction projects:

- Quality of discharge
- Effluent limits
- BMP selection, installation, and maintenance
- Visible or measurable erosion
- Recovery of offsite sediment
- Concrete washout
- Bulk storage of petroleum products
- Construction site dewatering
- Temporary stabilization
- Minimum storm size for BMPs
- Allowable discharges
- Sanitary facilities
- Requirements of other agencies

The requirements of the WYPDES Permit terms and conditions should be reviewed in its entirety by the Permittee prior to any construction-related activity.

11.2.3 City of Gillette Stormwater Program and Permit

Similar to the WYPDES Stormwater Program, the City requires a stormwater discharge permit for land disturbing construction activities. The City's Stormwater Program requires a Stormwater Permit (COG SW Permit) for any project that entails land disturbing or grading activities with a disturbance of 2,500 square feet or more in area.

These COG SW permits will require property owners, builders and construction site operators to:

1. Develop and implement a Stormwater Pollution Prevention Plan (SWPPP). The SWPPP outlines the BMPs or other control measures that will control stormwater discharges from the project site, ensuring the activities will not cause a violation of Wyoming Water Quality Standards (specific requirements of a SWPPP are detailed in Section 11.4);
2. Implement and maintain appropriate erosion and sediment control BMPs during construction;
3. Control waste such as discarded building materials, concrete truck wash out, chemicals, litter, and sanitary waste at the construction site that may cause adverse impacts to water quality; and
4. Implement a procedure to review site plans that incorporate consideration of potential water quality impacts (See Section 11.4.5).

11.2.4 Related Programs and Strategies

Other federal, state, and local programs and strategies which address the enhancement of stormwater quality and need to be considered when developing a SWPPP include:

- Total Maximum Daily Loads (TMDLs)
- NPDES Compliance Monitoring
- Green Infrastructure and Low Impact Development (LID), discussed in Section 12 of these Drainage Criteria

11.2.4.1 TMDLs

Thousands of surface waters are listed as impaired throughout the nation for various uses (recreation, drinking water, irrigation, aquatic life) due to excessive pollution from various sources. The most common pollutants coming from stormwater sources are sediment, pathogens, nutrients, and metals. Any water body determined to be impaired for its uses is listed on the state's 303(d) List and requires the development of a TMDL. A TMDL identifies the maximum pollutant loading, usually expressed as a concentration in water quality standards, that a waterbody can receive and still meet water quality standards. A TMDL specifies a pollutant allocation to specific point and non-point sources.

With the expansion of NPDES/WYPDES stormwater regulations to smaller municipalities and smaller construction activities, there has been increasing demand for more detailed quantification of stormwater allocations in TMDLs to facilitate implementation in NPDES/WYPDES permits. The trend recently is in development of more detailed stormwater source TMDL allocations, and TMDL implementation plans including BMPs. Also included is the requirement to develop implementation tools for translating TMDL allocations into NPDES/WYPDES stormwater permit requirements.

Currently, there are three 303d-listed water bodies within the City's watersheds with impairments to beneficial uses resulting from the following pollutants:

- Gillette Fishing Lake – *phosphorus and sediments*
- Donkey Creek – *E. coli*
- Stonepile Creek – *E. coli*

TMDL studies are in progress for all three. Any construction-related activities within the vicinity of these waterbodies need to provide BMPs that will minimize the transport of stormwater runoff pollutants from the project site to these receiving waters.

11.2.4.2 Compliance Monitoring

Another ongoing strategy for compliance involves monitoring surface water to support permitting compliance goals for the entire WYPDES program. Monitoring could be required for major and minor WYPDES facilities, pretreatment programs, biosolids, stormwater, oil and gas extraction/produced water discharges and Animal Feeding Operations (AFOs). Where a project has the potential to cause further impairment to a waterbody, the City may require a monitoring program to characterize the discharges from construction activities.

11.3 STORMWATER PERMITTING PROCESS

Construction and land disturbing projects require a Stormwater Permit from the City of Gillette. A COG SW Permit is required if any of the following conditions are present:

1. The site disturbance is greater than or equal to 2,500 sq. ft. Guidance for subdivisions and commercial sites is provided in Section 11.3.2. Single family lots are also required to obtain a COG SW Permit (see Section 11.3.3 for guidance and supporting Model SWPPP Drawings).
2. The site disturbance is less than 2,500 sq. ft. but meets one of the conditions:
   a. The site has been identified by the City as having a significant potential for erosion, based on site characteristics including topography.
   b. The site is known to contain contaminated soils on site or have a pre-existing condition warranting special care during construction.
   c. The site is within the designated 100-year floodplain of a drainageway.

The Stormwater Permit development process is illustrated in Figure 11-1.

11.3.1 Basis of Size Determination

The acreage used to determine the requirements for permit coverage is the total area of disturbance for the project.

To determine the total area of disturbance for a project:

1. Define the construction limits of the project
2. Include all construction areas involved with the project, which may include private land and public land or right-of-way.
3. Include all areas to be used for equipment or material storage.
4. Calculate the disturbance area enclosed by the construction limits

If the project is within public ROW, calculate the disturbed area as follows:

Area = (ROW Width x Project Length) + Area Calculated Per #3 above

Figure 11-1 provides a schematic of the City's Stormwater Permit Development Process for construction activities, including detail on when a SWPPP is expected to be developed. As identified in this figure, once a project has been identified and area of disturbance calculated, the Applicant (Owner/Developer/Builder) will follow one of two procedures for obtaining a permit. Sections 11.3.2 and 11.3.3 provides additional information regarding permit requirements.

11.3.2 Subdivisions and Commercial Sites

For subdivisions and commercial projects disturbing areas 2,500 sq. ft. and greater, a SWPPP meeting the requirements of Section 11.4 shall be prepared. A preliminary SWPPP shall be submitted to the City along with the Preliminary Plat Submittal. Once the City has reviewed and approved this information, the Applicant will incorporate/update the SWPPP as needed and submit a Final SWPPP with the Final Construction Plan submittal. Once the City has reviewed this information, the Applicant will submit a COG Stormwater Permit application along with applications for a Permit to Construct or a Grading Permit to the City. If the project involves disturbance of 5 acres or more, the Applicant shall submit a Notice of Intent (NOI) to the WYDEQ and provide a copy of the NOI with the COG Stormwater Permit application.

11.3.3 Single Family Residential Projects

For single family residential projects disturbing 2,500 square feet or more, the City also requires a COG SW Permit and a SWPPP. If a Building Permit is required for the project, the owner/builder shall determine whether the project involves site grading activities disturbing 2,500 square feet or more. If site grading activities disturb 2,500 square feet or more the owner/builder may develop a SWPPP using one of the residential sites templates, Section 11.4. The SWPPP shall be submitted to the City along with the Building Permit Application. The City has developed two Model SWPPP Drawings for residential sites (see Figures 11-4 and 11-5), which may be adapted to the site plan of individual single family residential building sites by the Owner/Builder.

11.3.4 Construction Requirements

Once the Permittee has obtained a Building Permit and/or a Permit to Construct, and COG SW Permit, the Permittee for the site shall implement the SWPPP before construction begins. The City shall inspect the site and, if the installation of the BMPs is in conformance with the SWPPP, construction will be allowed to begin. The Permittee will be responsible for periodic inspection and maintenance of the BMPs, and the site will be subject to periodic inspections by the City, see Figure 11-2. It is the responsibility of the Permittee or their delegated SWPPP Administrator to maintain the BMPs according to the approved SWPPP.

Should the Permittee fail to properly install or maintain BMPs, or otherwise not comply with the terms and conditions of the COG SW Permit and/or WYDEQ Stormwater Discharge Permit, the City may issue a Notice of Deficiency or a Notice of Violation to the Permittee, and the Permittee shall be subject to enforcement proceedings. The environmental inspection and enforcement process for the City's compliance with the Stormwater Discharge Permit for construction activities is illustrated in Figure 11-3.

11.3.5 Transfer of Permits

The City of Gillette Stormwater permit can be transferred by completing a City of Gillette Stormwater Permit Notice of Transfer form and filing it with the City's Engineering and Building Department. For projects that require a WYPDES permit, the WYDEQ Notice of Transfer and Acceptance (NOTA) form must be submitted and a copy provided to the City prior to transferring responsibility of the WYPDES permit

11.3.6 Termination of Permit

When construction is complete and final stabilization measures installed, the Permittee shall notify the City and the City shall perform a final inspection. A Certificate of Termination (COT) to discontinue coverage under the Stormwater Permit shall be issued when final stabilization measures are in place and approved by the City. The construction site must be "finally stabilized" as defined in these Drainage Criteria.

Figure 11-1
Stormwater Permit Development Process

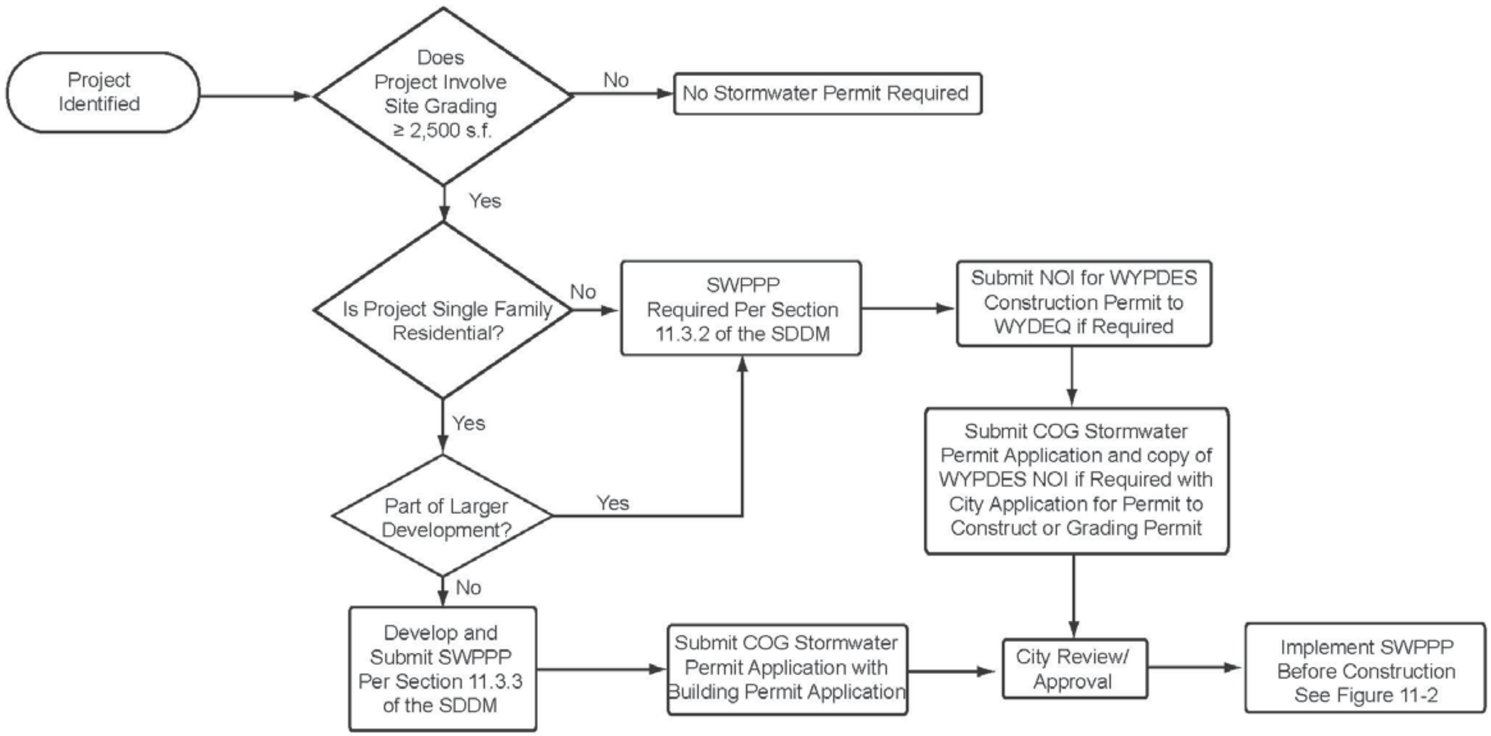
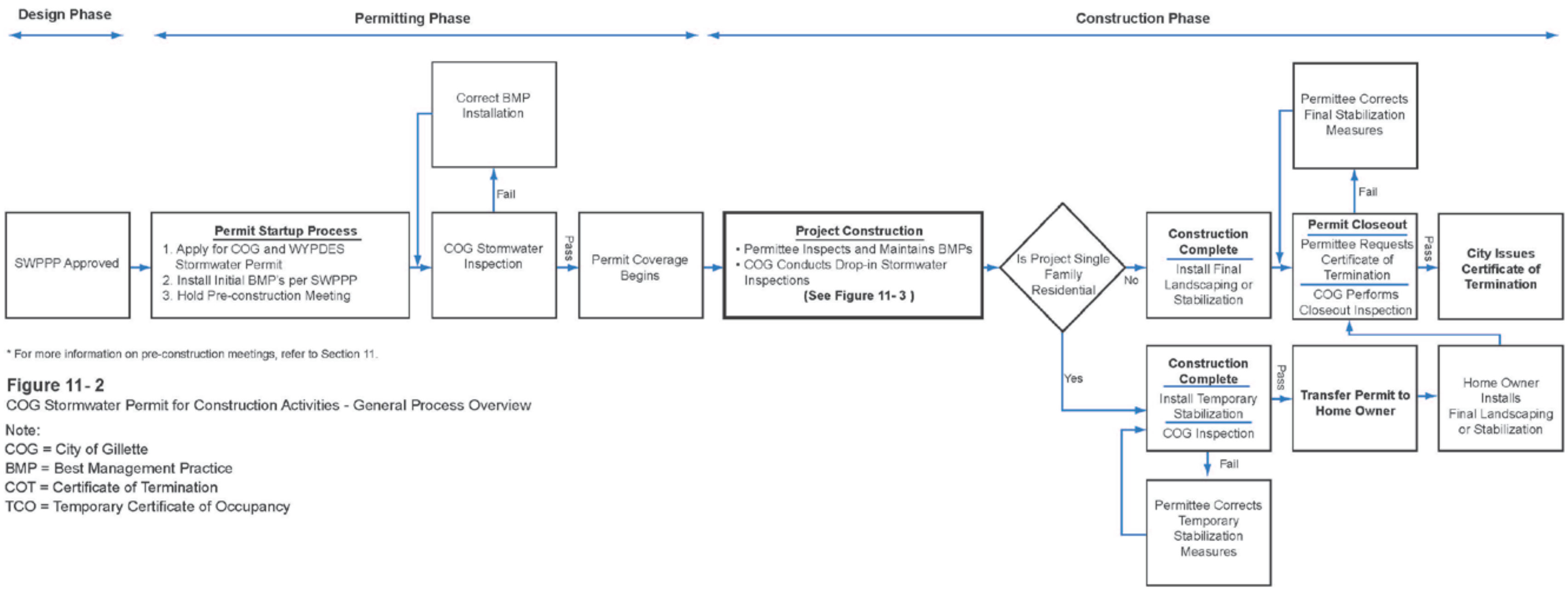


Figure 11 - 1
Stormwater Permit Development Process

Notes:
SWPPP = Stormwater Pollution Prevention Plan
SDDM = Storm Drainage Design Manual
NOI = Notice of Intent
COT = Certificate of Termination
COG = City of Gillette

Figure 11-2
COG Stormwater Permit for Construction Activities – General Process Overview



* For more information on pre-construction meetings, refer to Section 11.

Figure 11-2
COG Stormwater Permit for Construction Activities - General Process Overview

Note:
COG = City of Gillette
BMP = Best Management Practice
COT = Certificate of Termination
TCO = Temporary Certificate of Occupancy

Figure 11-3
Environmental Inspection and Enforcement Process for COG Stormwater Permit for Construction Activities

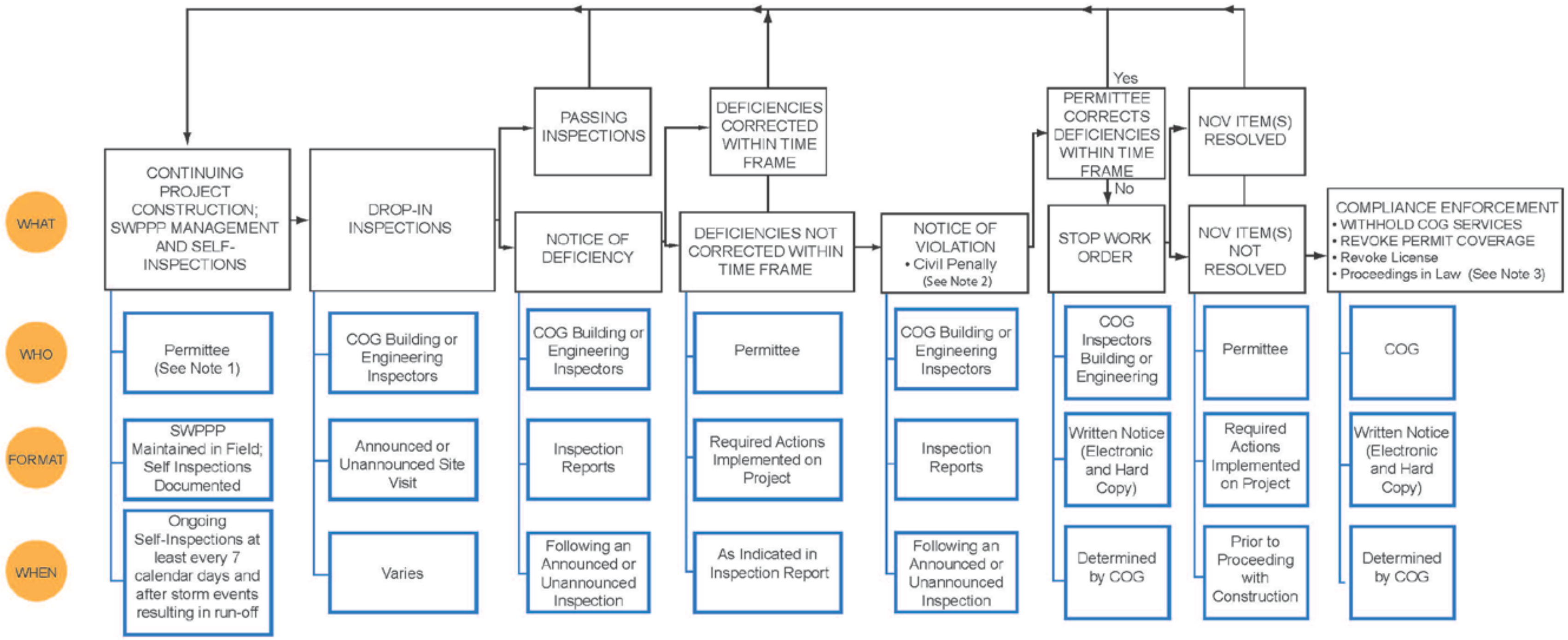


Figure 11 - 3
Environmental Inspection and Enforcement Process for COG Stormwater Permit for Construction Activities

Notes:

1- Responsible party is listed here. COG does not preclude responsible party from contracting/delegating some, all, or none of its responsibilities, but Permittee remains responsible for Permit compliance whether or not Permittee elects to delegate certain responsibilities.

2- Any violation of Article 6 of the Gillette City Code is subject to a minimum fine of $750/day and any other penalty as prescribed under Section 16 -1 of the Gillette City Code

3- Permittee may appeal. Submitting an appeal does not relieve Permittee's obligations and timelines for action items. Appeals will be evaluated within 48-72 hours by the COG Board of Examiners.

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Figure 11-4
Model S.W.P.P.P. Drawing A

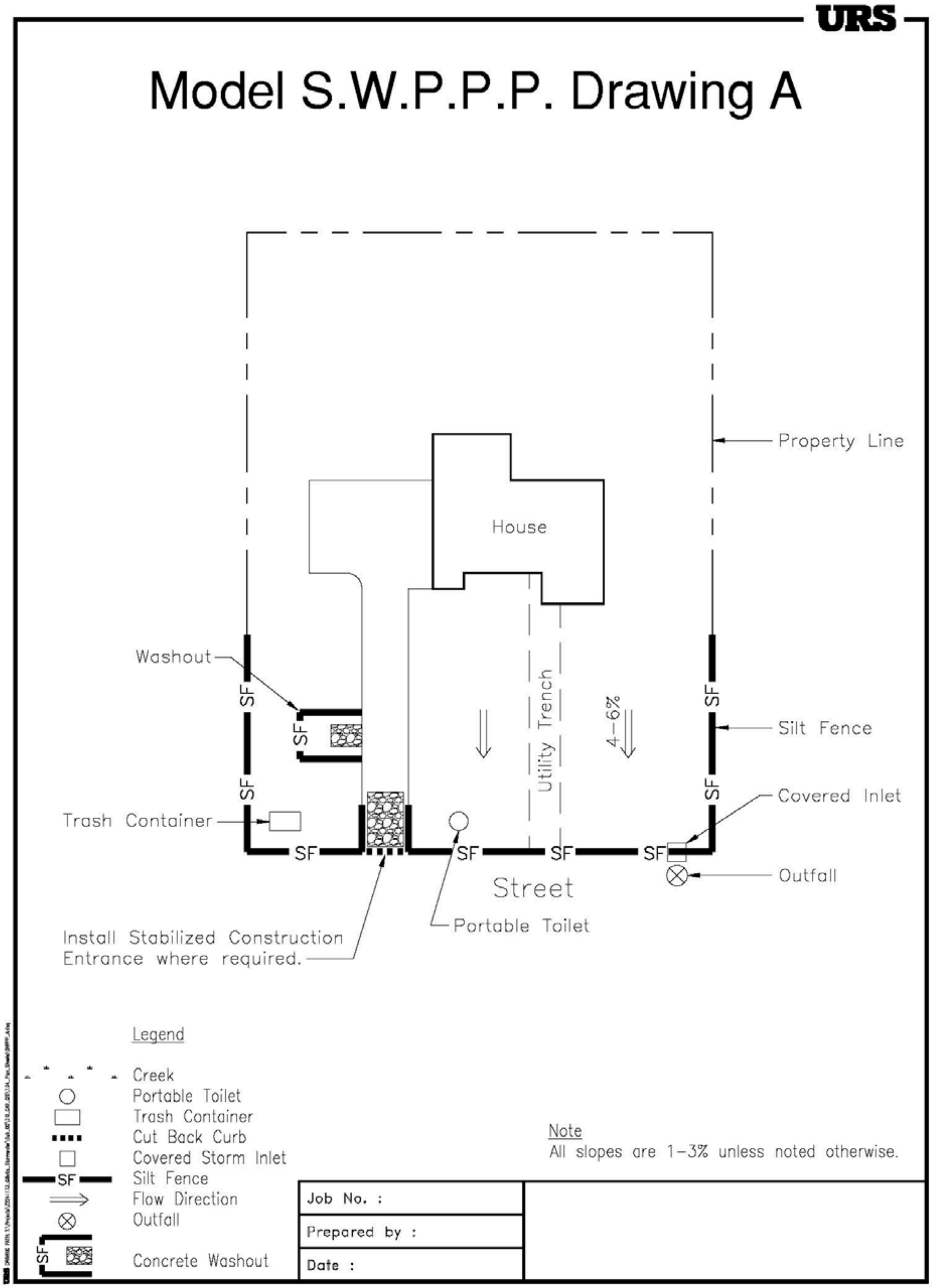
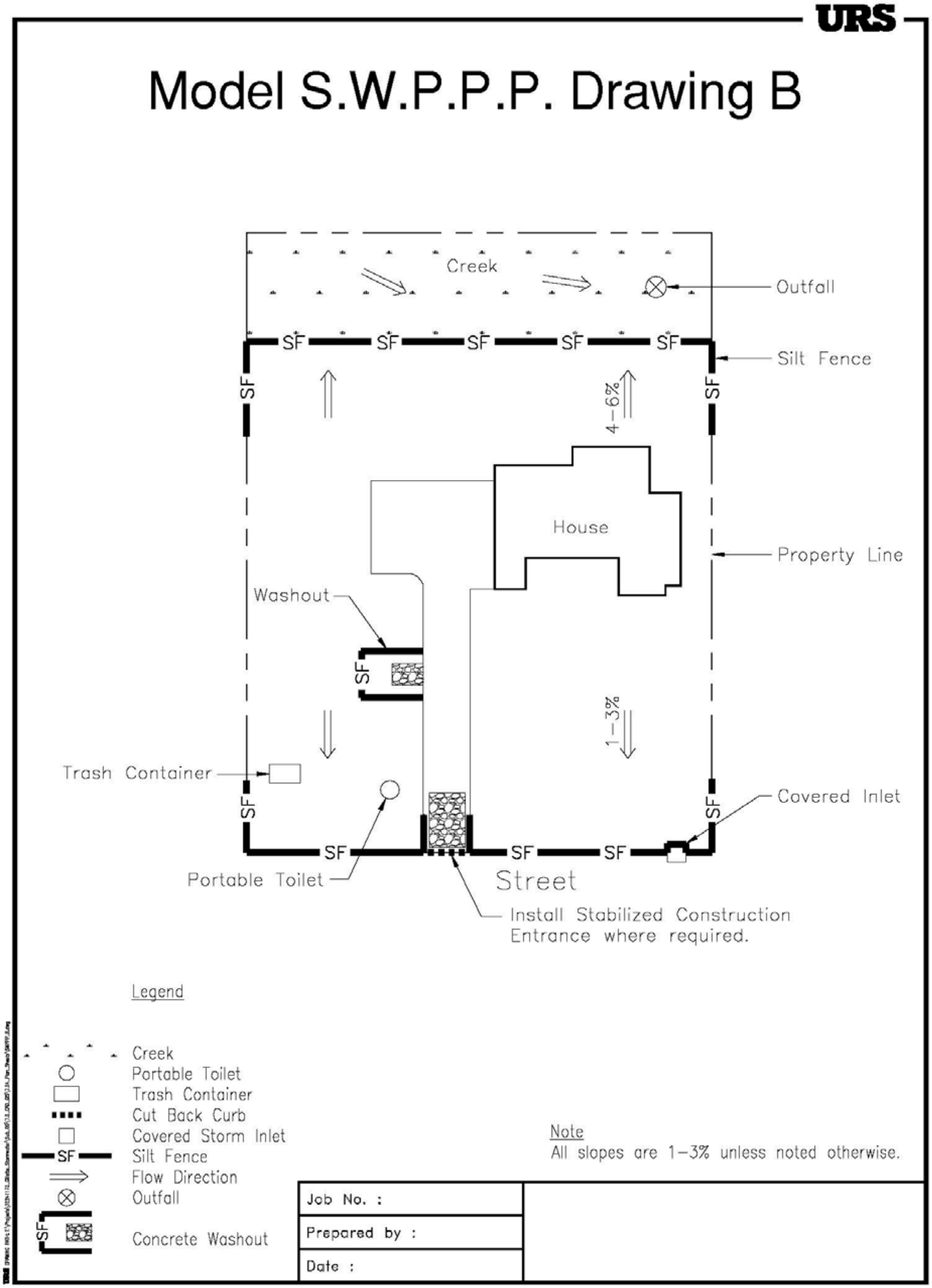


Figure 11-5
Model S.W.P.P.P. Drawing B



11.4 STORMWATER POLLUTION PREVENTION PLAN (SWPPP)

All projects that disturb 2,500 square feet or more are required to implement BMPs to control erosion, sedimentation, and pollutant laden stormwater discharges during construction activities according to an approved SWPPP. This requirement applies to all new land disturbing activities or redevelopment projects and includes grading projects or demolition activities where there is to be any excavation, trenching or other disturbance of the existing ground surface.

The City requires that a SWPPP be submitted along with the COG SW Permit Application, the Permit to Construct (Final Construction Plan) and/or the Building Permit Application, as applicable by project. The SWPPP shall be site specific, with each new construction project necessitating its own SWPPP. The SWPPP must be implemented immediately and continuously throughout the duration of the construction activity until the site is finally stabilized. After construction begins, a copy of the SWPPP should also be readily available on the site for inspection purposes.

The SWPPP shall include:

- Identification of a SWPPP Administrator
- A Site Description
- A Site Map
- Type and Location of proposed BMPs, and perimeter controls.

Additional detail for each of the identified SWPPP components is described in the following paragraphs.

11.4.1 SWPPP Administrator

Each SWPPP shall identify a specific individual or individuals within the facility organization that are responsible for developing the stormwater SWPPP and assisting the facility manager in its implementation, maintenance, and revision. The SWPPP shall clearly identify the responsibility of plan administration, either by name or job title.

11.4.2 Site Description

The site description includes:

- A brief description of the existing vegetation at the site and an estimate of the percent of vegetative ground cover. Always maintain as much vegetation as possible during construction to minimize erosion.
- A brief description of the construction activity.
- Nature of the construction activity;
- Location(s) of equipment storage, cleaning and maintenance areas and activities;
- Points of ingress and egress to the construction site;
- Material loading and unloading areas;

- Storage areas, including construction materials, building materials and waste materials; and
- Materials, equipment, or vehicles that may come in contact with stormwater.
- The proposed sequence of major activities and a planned completion date.
- Describe the intended sequence of major activities which disturb soil on the site. Major activities shall include, but are not limited to, all excavation and backfill operations as well as grading operations.
- Specify the anticipated starting and completion dates of the site grading and/or construction sequence, including the installation and removal time periods of erosion and sediment control measures, and the time of exposure of each area prior to the completion of temporary erosion and sediment control measures.
- An estimate of the total area of the site and an estimate of the area expected to undergo clearing, excavation or grading, including off-site borrow areas, access roads, and staging/storage areas. Explain whether clearing/grading activities will be phased.
- The location and description of any other potential pollution sources including, but not limited to, vehicle fueling areas, storage areas for fertilizers, chemicals, used oil, concrete washout or paint. Explain whether these sources will be disturbed in construction operations and estimate the volume of runoff from each area. Provide BMPs to match volume and velocity of expected flows, and include a list any non-sediment types of pollutants.
- Identify the drainageway, surface or water body that will receive a stormwater discharge from the construction activity and the size, type, and location of any outfalls. If the discharge is to the MS4, identify the location of the storm drain outfall and the drainageway or water body that will receive stormwater discharges from the municipal outfall.
- Implementation of specific BMPs for a project discharging to receiving waters with a TMDL.

11.4.3 Site Map

Any submitted site maps must be readable, with clearly identified pollutant sources and erosion controls. Maps shall identify drainage patterns, soils, vegetation, surface water bodies, and steep or unstable slopes. Site maps do not have to be certified or professionally drafted, but they must demonstrate clear planning for erosion and sediment control on the site, have an appropriate scale, and information should be readily retrievable. It is acceptable to submit multiple maps.

The SWPPP shall provide a site map or maps that indicate, at a minimum:

1. Construction site boundaries.
2. All areas of soil disturbance.
3. The location of surface waters of the state as defined in the WYPDES Permit, including springs, streams, wetlands, ponds and any defined drainageways that receive stormwater discharges from the site.

4. Areas used for storage of building materials, soils, wastes, fuel, and areas used for concrete washout.
5. Locations of proposed or existing stormwater controls.
6. Site topography and/or stormwater drainage patterns and discharge points (outfalls).
7. Where proposed as part of the permitted project, include site maps for off-site concrete/asphalt batch plants, borrow areas and/or fill material disposal areas, and equipment/materials staging and storage areas.

11.4.4 Best Management Practices (BMPs)

The SWPPP shall include a narrative description of appropriate erosion and sediment controls and BMPs that will be implemented before, during, and after construction. The SWPPP shall clearly describe the relationship between the phases of construction and the implementation and maintenance of BMPs.

Note that for single family residential projects that have only one phase of construction, the City has developed two drawings for guidance in selecting BMPs (Figure 11-4 and Figure 11-5).

The description of controls shall address the following components:

1. EROSION AND SEDIMENT CONTROL. An erosion and sediment control plan shall identify appropriate control measures for each major phase of construction. The Permittee shall be responsible for selection, design and layout of appropriate erosion and sediment control BMPs. Design guidance is provided in Section 11.5.
2. CONSTRUCTION SITE DEWATERING. The SWPPP must specify BMPs for discharges from construction site dewatering. Discharges must meet the conditions specified in the WYPDES Permit, including the use of settling or filtration techniques as appropriate and the use of velocity dissipation devices at the outlet.
3. POST-CONSTRUCTION CONTROLS. A description of the temporary stabilization measures that will be implemented after construction is complete and until final stabilization is achieved.
4. SOURCE CONTROLS. The SWPPP shall describe BMPs used in day-to-day operations on the project site to reduce the potential of pollutants in stormwater runoff, such as:
    a. Good housekeeping BMPs to maintain a clean and orderly facility. At a minimum, the SWPPP should address control of litter, debris, chemicals, fertilizers and sanitary wastes, and measures to remove sediment that has left the construction site.
    b. Bulk storage of petroleum products. The SWPPP shall describe:
        i. Practices that provide adequate protection so as to contain all spills and prevent any spilled materials from entering waters of the state or municipal storm sewer systems.
        ii. Practices for addressing a spill including methods of handling and disposing spilled products and contaminated soils.

c. The facility spill prevention control and countermeasures (SPCC) plan may be referenced in the SWPPP as fulfillment of this requirement. The SPCC should be attached to the SWPPP if it is referenced.

d. Concrete washout. Concrete wash waters shall be contained and shall not be discharged to surface waters of the state or municipal storm drains. The SWPPP shall describe appropriate BMPs to control stormwater pollution from portable concrete or asphalt batch plants covered under this permit.

e. Construction site phasing. Construction site phasing minimizes soil erosion through a somewhat more complex construction process. Only one portion of a site is disturbed at any one time to construct the infrastructure necessary to complete that phase. Subsequent phases are not started until earlier phases are substantially completed and exposed soils are mostly stabilized. This "just-in-time" construction practice can dramatically reduce disturbed soil exposure times and resulting erosion problems (Center for Watershed Protection, found at: http://www.cwp.org). For example, indicate which controls will be implemented during each stage of construction, such as clearing and grubbing necessary for perimeter controls, initiation of perimeter controls, remaining clearing and grubbing, road grading, storm drain installation, final grading, stabilization, and removal of control measures.

5. MAINTENANCE. All BMPs identified in the SWPPP must be maintained in effective operating condition. The plan must indicate, as appropriate, the intervals or conditions upon which BMPs shall be inspected and maintained. Maintenance shall occur whenever periodic inspections identify BMPs that are not operating effectively. Maintenance shall be accomplished as soon as is practical after major storm events.

6. INSPECTIONS. The plan must provide for site inspections to monitor the condition of stormwater outlets and the effectiveness of BMPs. The permittee shall ensure that personnel conducting site inspections are familiar with the requirements of the SWPPP and proper operation and maintenance of all implemented BMPs. All inspections shall be conducted in accordance with Part 9 of the DEQ's General Permit to Discharge Stormwater.

7. SIGNATURE. All SWPPPs submitted to the City of Gillette must include the certification language stated in Part 10.7.4 and must be signed in accordance with Part 10.7 of the DEQ's General Permit to Discharge Stormwater.

11.4.5 SWPPP Amendments

The Permittee shall modify the plan whenever there is a change in design, construction, operation, or maintenance that changes the potential for the discharge of pollutants, if it proves ineffective in eliminating or minimizing pollutants present in stormwater runoff from the site. The most current version of the SWPPP must be retained on site or located as described below in Section 11.4.6. The SWPPP may be reviewed during site inspections.

11.4.6 Plan Retention

The SWPPP shall be retained at the construction site during active construction. When the project is shut down for the season or at the completion of construction the SWPPP may be kept offsite. Requirements for maintaining the SWPPP at a construction site are as follows:

1. The location of an off-site SWPPP must be posted on site. The posting shall note the location of the SWPPP, a contact phone number and the stormwater authorization number.
2. If posting the offsite location at the construction site is impractical due to remote location or the facility is impractically large for a posting, the operator may send a brief letter to the DEQ Storm Water Coordinator and the City of Gillette specifying the site authorization number, location of the SWPPP and a contact telephone number for a person with access to the SWPPP.
3. For all SWPPPs the Permittee must provide reasonable local access to the plan during normal working hours. The Permittee shall make the SWPPP available upon request to the Administrator or agent thereof; any federal, state or local agency; interested members of the public; local government officials; or to the operator of a municipal separate storm sewer receiving discharges from the site.

11.4.7 Employee Training

The site operator's personnel shall be informed of erosion and sediment control, spill response, good housekeeping, and materials management practices.

11.4.8 During Construction

Construction cannot begin until perimeter controls are in place and inspected by the City of Gillette. After construction begins, it is vital that the SWPPP be followed. The BMPs must be implemented, inspected and maintained according to the written schedule in the SWPPP. If any changes in the plan occur, they should be documented and the City must be notified.

11.4.9 Complete Final Stabilization

When all land disturbing activities and final stabilization of the construction site are complete, a request for a Certificate of Termination (COT) shall be made to the City. Final stabilization is defined in the Definitions at the front of these Drainage Criteria. After inspection by the City, the permit will be closed.

11.5 BMP DESIGN AND SELECTION

Erosion and sediment control practices are required for all land disturbing projects affecting sites 2,500 square feet or more. All land disturbing projects shall be designed and managed to minimize the loss of soils from disturbed areas and the deposition of eroded materials downstream during and after construction. Designs shall also include the restoration and stabilization of disturbed areas to minimize the loss of soils after construction.

Tables 11-1 and 11-2 provide a list of recommended structural and source control BMPs for projects within the City; however, the Permittee can use additional BMPs with supporting documentation that it is applicable to the project. Note that the selection of the BMPs is up to the Permittee and his delegated design staff or contractor preparing the SWPPP. It is important to understand the specific site conditions which dictate the proper selection, installation and maintenance requirements for each BMP.

When preparing the SWPPP, it is important to minimize exposure, ensure physical space for BMPs, and incorporate the BMPs into other construction activities where possible. Plan how often BMPs will be inspected for maintenance needs, and consider the cost, not only of implementation but also on-going maintenance costs.

Select the BMPs to be used to match the pollutant of concern. Include a description of when and how BMPs shall be incorporated into the work. Include any engineering calculations in the Drainage Report supporting these selections. Technical specifications of BMPs, materials, and resources shall be provided with the SWPPP.

General design criteria for the selection of these BMPs include, but are not limited to the following:

- Stormwater BMPs are expected to withstand and function properly during precipitation events up to a 2-year, 24-hour storm event. The 2-year, 24-hour storm event in Gillette is 1.6 inches (see Table 2-3 of these Drainage Criteria).
- List structural and source control BMPs. These BMPs can be listed in bullet, table or phase form, with a clear description accompanying them. Be sure to list BMPs used for planning, the major construction activities, and all post-construction activities.
- Describe other proposed controls such as sediment tracking, fuel storage and handling, sanitary wastes, trash collection and disposed controls for potential pollutants that may be brought onto the site.

11.5.1 Structural BMPs for Construction Sites

Table 11-1 provides a summary of the recommended structural BMPs to be used for sediment and erosion control on land disturbing projects within the City. The three general categories of BMPs include:

- Sediment control BMPs. Sedimentation occurs when soil is eroded and transported from its original location. The goal of sediment control is to prevent sediment from leaving the construction site and, specifically entering surface waters. Every SWPPP shall describe adequate BMPs to achieve sedimentation control within site perimeters.
- Erosion control BMPs. The goal of erosion prevention is preventing soil (or sediment) movement and keeping it at its original location within the construction site. Each SWPPP shall provide BMPs for erosion prevention and containment within site perimeter.
- Temporary erosion protection BMPs. Temporary stabilization (such as cover crop plantings, mulching or erosion controls blankets, and surface roughening) for exposed soil areas where land disturbing activities have temporarily ceased shall be installed

whenever practicable in areas where further work is not expected for 28 days or more. Areas to be protected include graded slopes, ditches, berms and soil stockpiles.

Table 11-1
Recommended Structural BMPs

BMP	Purpose
Stabilized Construction Entrance	Sediment Control
Storm Drain Inlet Protection	Sediment Control
Stormwater Inlet/Outlet Protection	Sediment Control
Check Dam	Sediment Control
Silt Fence	Sediment Control
Wattles	Sediment Control
Filter Berms	Sediment Control
Sediment Basin	Sediment Control
Sediment Trap	Sediment Control
Diversion Swale/Berm	Sediment Control
Temporary Slope Drain	Erosion Control
Erosion Control Blankets	Erosion Control
Surface Roughening	Erosion Control
Temporary Seeding and Planting	Erosion Control
Mulching	Erosion Control
Soil Retention Measures	Erosion Control
Wind Erosion Controls	Erosion Control
Steep Slope Terracing	Erosion Control

A fact sheet for each of the structural BMPs listed in Table 11-1 is provided in Section 11.6.

11.5.2 Source Control BMPs

Source control BMPs are intended to prevent or reduce the contamination of stormwater runoff from a variety of different sources or activities, by reducing pollutant generation. Preventing and controlling the sources of pollutants requires a change in behavior.

The type of source control BMPs discussed in Section 11.5.4 below can be applied to various different types of sites. Because they rely on actions and not structures, source control BMPs must be implemented constantly and repetitively over time.

The main objectives of using source control BMPs are to:

1. Reduce or eliminate the pollutants that impact water quality at their source, thus reducing the need for structural control requirements. For example, source control BMPs implemented at an industrial site may result in elimination or reduction of the introduction of oils and greases into the stormwater. This could result in the better efficiency of an infiltration basin or the elimination of the need for additional treatment for oils.

2. Address water quality concerns that are not cost effectively handled by structural controls.

11.5.2.1 Advantages of Source Control BMPs

Source control BMPs are measures that prevent or limit the entry of pollutants into stormwater at the source. Prevention is desirable and can be cost effective, because it minimizes pollution in the first place and by careful management and operations, thereby reducing amounts that need to be removed by subsequent stormwater treatment.

The advantages of source control BMPs are:

1. Improved stormwater quality.
2. The volume of sediment, debris, oils, chemicals and other pollutants deposited in receiving surface waters is reduced.
3. Frequency of needed maintenance of structural controls is reduced.
4. Additional benefits to air quality, ground water quality, and solid waste control are realized.
5. The public awareness of stormwater quality issues and problems is increased, and involvement in solutions is heightened.
6. Most are simple to understand, make good sense and require only a modification of existing practices
7. Implementation can occur rapidly.
8. They do not require major capital construction financing.

11.5.2.2 Source Control BMP Selection and Use Guidelines

Most source control BMPs are applicable for use in residential, commercial, and industrial areas. This can be said for newly developing areas, recently developed areas, and old neighborhoods as well.

The source control BMPs included in the following section are targeted at construction activities for developed areas and industrial/commercial sites. Most source control BMPs rely primarily on public/staff education, and procedural changes and possible enforcement programs established by the City. In selecting the appropriate source control BMP each Owner/Project Manager needs to evaluate its specific project, the current land-use condition and the potential for removal of pollutants offered by each of these practices. The specifics for selection and use of each source control BMP effort are described for each management practice are described below. The general category in which source control BMPs are classified for construction and municipal operations is Good Housekeeping. Overall, Good Housekeeping BMPs provide a proper location, storage and maintenance of a particular construction material in addition to the incorporation of planning and administration of construction projects to minimize erosion and sedimentation.

It is recommended that as part of the overall planning process an evaluation be made of the potential sources of pollutants during construction and the best means of addressing and mitigating them. For example, if the problem is oil and grease in the runoff from a heavy equipment use, then source control controls dealing with spill prevention may be appropriate. Thus, once the source or activities are determined the list of BMPs should be consulted to determine the most applicable practices.

A fact sheet for each of the source control BMPs listed in Table 11-2 is provided in Section 11.6.

Table 11-2
Source Control BMPs

BMP	Purpose
Concrete Washout	Good Housekeeping/Source Control
Waste Management	Good Housekeeping/Source Control
Use of Pesticides, Herbicides and Fertilizers	Good Housekeeping/Source Control
Illicit Discharge Controls	Good Housekeeping/Source Control
Good Housekeeping	Good Housekeeping/Source Control
Preventative Maintenance	Good Housekeeping/Source Control
Spill Prevention and Response	Good Housekeeping/Source Control
Stock Pile Management	Good Housekeeping/Source Control

11.5.3 SWPPP Checklist (Note that the Permit referenced in this table is the WYPDES General Construction Stormwater Permit)

Table 11-3 outlines a checklist that can be used to ensure that the SWPPP includes information relevant to the WYPDES Construction Stormwater Permit requirements.

Table 11-3
SWPPP Checklist

Complete?	Description	Gillette Stormwater Permit	WYPDES Permit Part
	Identify individual(s) responsible for implementing, maintaining, and revising the SWPPP, including contact information.	11.4.1	7.2.1
	Site Description		7.2.2
	Describe the nature of the construction activity.	11.4.2	7.2.2.1
	Describe the proposed sequence of major activities and completion date.	11.4.2	7.2.2.2
	Estimate of the total area of the site disturbance including off site borrow areas, access roads, and staging/storage areas.	11.4.2	7.2.2.3
	Describe existing site vegetation and percentage of ground cover.	11.4.2	7.2.2.4
	Describe any other potential pollution sources.	11.4.2	7.2.2.5
	Locations of drainages that may receive stormwater runoff from this construction activity.	11.4.2	7.2.2.6

Table 11-3
SWPPP Checklist

Complete?	Description	Gillette Stormwater Permit	WYPDES Permit Part
	Site Map		7.2.3
	Construction site boundaries.	11.4.3	7.2.3.1
	All areas of soil disturbance.	11.4.3	7.2.3.2
	The location of surface waters of the state as defined in Part 2.20 of the WYPDES General Permit.	11.4.3	7.2.3.3
	Areas used for storage and concrete washouts.	11.4.3	7.2.3.4
	Locations of existing or proposed BMPs.	11.4.3	7.2.3.5
	Site topography or stormwater drainage patterns.	11.4.3	7.2.3.6
	Where included as part of the permitted project, include site maps for offsite concrete/asphalt batch plants, borrow areas and/or fill material disposal areas, and staging and storage areas.	11.4.3	7.2.3.7
	Best Management Practices (BMPs)		7.2.4
	Erosion and Sediment Controls		7.2.4.1
	Erosion prevention BMPs to be used on site.	11.4.4	7.2.4.1.1
	Sediment control measures to be used on site.	11.4.4	7.2.4.1.2
	Temporary erosion protection measures that will be used in exposed soil areas where activities have permanently or temporarily ceased.	11.4.4	7.2.4.1.3
	BMP selection, installation, and maintenance procedure.	11.4.4	7.2.4.1.4
	Are BMPs designed to withstand and function properly during a 2-year, 24 –hour storm event?	11.5	7.2.4.1.5
	List construction site dewatering BMPs that will be used. BMPs must meet Part 8.8 conditions.	11.4.4	7.2.4.2
	Describe the temporary stabilization measures that will be implemented after construction is complete and until final stabilization is complete.	11.4.4	7.2.4.3
	Source Controls		7.2.4.4
	Describe good housekeeping BMPs that will be used to address litter, debris, chemicals, fertilizers, sanitary wastes, and sediment that may leave the construction site.	11.4.4	7.2.4.4.1
	Describe BMPs for petroleum product bulk storage and spill management.	11.4.4	7.2.4.4.2
	Describe on site concrete washout facilities.	11.4.4	7.2.4.4.3
	List appropriate BMPs that will be used to control stormwater pollution from concrete or asphalt batch plants covered under this permit.	11.4.4	7.2.4.4.4
	Does the plan indicate the conditions or intervals upon which BMPs shall be maintained?	11.4.4	7.2.4.5

Table 11-3
SWPPP Checklist

Complete?	Description	Gillette Stormwater Permit	WYPDES Permit Part
	Provide a schedule of inspections for BMPs as described in Part 9 of the WYPDES General Permit.	11.4.4	7.2.4.6
	All SWPPPs must be signed in accordance with Part 10.7 of the WYPDES General Permit.	11.4.4	7.2.4.7

11.6 STRUCTURAL AND SOURCE CONTROL BMP FACT SHEETS

The structural and source control BMP fact sheets are divided into sections based on the type of measure and are intended as a guide for determining practical applications for a project site. Each fact sheet includes:

- Description of the BMP.
- Design criteria.
- Advantages and limitations of the specific BMP.
- Maintenance and cost information where there was available data.
- Stormwater management suitability (providing guidance on the appropriate applications of that BMP).
- Implementation considerations [providing insight on land requirements, capital costs, and potential maintenance burden(s)].

An “L”, “M”, or “H” is placed in front of each of the categories indicating the relative amount of land required, cost, and/or maintenance needs for each of the BMPs listed. For a low requirement, an “L” is used; for a moderate requirement, an “M” is used, and for a high requirement, an “H” is used. *Note, the determination of “L”, “M”, and “H” are based on the relative (qualitative) comparison with the other listed BMPs in this section only.*

It is important to have supporting documentation and/or justification for the selection of BMPs for a project. BMP selection is dependent upon several factors, including, but not limited to:

- Project type (construction, new/existing land disturbing activity; road; good housekeeping)
- Pollutants of concern (e.g., sediment, phosphorus, *e. coli*, metals, salts)
- Site Conditions (precipitation, topography, soils)
- Proximity to surface water bodies with water quality standards
- Proximity to adjacent land owners, agencies, and jurisdictions
- Objectives of the City’s Stormwater Program
- Objectives of the City’s Long-Term Stormwater/Environmental Goals including, but not limited to, Low Impact Development/Green Initiatives

These factors need to take into consideration the regulatory status of the City, as some permits may specify numeric water quality goals for pollutant removals from BMPs.

Guidance on the evaluation, selection and use of source control BMPs as it applies to construction activities in urban areas is also presented in the following source control fact sheets.

Stabilized Construction Entrance

Construction BMP

Description: Stabilized construction entrance consists of a stone-stabilized pad located at any point where traffic will be leaving a construction site to a public roadway.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Applicable to any location where construction traffic leaves the site to an existing paved road.
- If designed to be a high-traffic entrance, it should be wide enough for the passage of two vehicles at the same time with room to spare.

ADVANTAGES / BENEFITS:

- Entrance stabilization may increase the aesthetics of construction site.

DISADVANTAGES / LIMITATIONS:

- Narrow/limited amount of area for passage of two vehicles.
- Availability of material/rock/stone.

MAINTENANCE REQUIREMENTS:

- Inspect and repair/replace stone and gravel at entrances.
- Sweep errant soil from public street immediately for proper disposal.
- Periodically remove the sediment from traps.
- Remove gravel and filter fabric at the end of construction.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Storm Drain Inlet Protection

Construction BMP

Description: Storm drain inlet protection consists of permeable barriers that help prevent soil and debris from site erosion from entering storm drain inlets. Inlet protection can be constructed from rock socks, sediment control logs, silt fence, block and rock socks, or other materials approved by the City. Area inlets can also be protected by excavating around the inlet to form a sediment trap.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- These controls should be installed before any soil disturbance in the drainage area.
- Should be used in drainage areas less than 1 acre.
- Filter fabric fence inlet protection is appropriate in open areas is subject to sheet flow and for flows not exceeding 0.014 m^3/s ($0.5ft^3/s$).
- Proprietary products may be used with the approval of the City.

ADVANTAGES / BENEFITS:

- Allows storm drains to be used during even the early stages of construction.

DISADVANTAGES / LIMITATIONS:

- Stormwater drop inlet protection measures are not used as stand-alone sediment control measures.
- These protection measures are practical for relatively low-sediment, low-volume flows.
- Can be an impediment to traffic.

MAINTENANCE REQUIREMENTS:

- Frequent maintenance is required.
- Inspect inlet protection devices before and after a storm.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
	Streambank Protection
x	**On-Site Flood Control**
x	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*
High Density/Ultra-Urban: *Yes*
Drainage Area: *N/A*
Soils: *N/A*
Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Stormwater Inlet Outlet Protection

Construction BMP

Description: Stormwater outlet protection consists of riprap or structurally lined aprons or other acceptable energy dissipating devices placed at the outlets of culverts, storm drain pipes or paved channel sections.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Install at all pipe, interceptor dike, swale or channel section outlets where velocity of flow may cause erosion at the pipe outlet.

DISADVANTAGES / LIMITATIONS:

- Outlet protection may be unsightly.
- This may cause sediment removal problems, necessitating removal of the outlet structure itself.
- Loose rock may be washed away during high flows.
- Grouted riprap may break up in areas of freeze and thaw.

MAINTENANCE REQUIREMENTS:

- Inspect temporary measure prior to predicted storm events, and as soon as possible after storm events, and regularly (once per week) during the construction season.
- Inspect apron for displacement of dissipation devices and/or damage to the underlying fabric and repair as needed.
- Inspect for scour beneath the dissipation devices and around the outlet. Repair damage to slopes or underlying filter fabric immediately.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
x	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Check Dams

Construction BMP

Description: Check dams are small, temporary barriers constructed across a swale or channel carrying stormwater from a disturbed area. They are generally constructed of straw bales, gravel or rock.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Straw bales - maximum of 0.25 acres per 100 feet of fence length with a 2:1 or shallower slope and 100 foot slope length.

ADVANTAGES / BENEFITS

- Temporary control measure.
- Large particle removal.
- Proper installation may reduce piping.

DISADVANTAGES / LIMITATIONS:

- Concentrated or high velocity flows.
- Straw bale barriers are attractive to livestock and wildlife and degrade quickly, which will shorten barrier life.

MAINTENANCE REQUIREMENTS:

- Regular maintenance is necessary to repair breaks and breaches.
- Sediment removal may also be necessary.
- Livestock and wildlife may find hay bales attractive necessitating more frequent replacement.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Silt Fence

Construction BMP

Description: A silt fence is a temporary linear sediment barrier of permeable, woven geotextile fabric designed to intercept and slow the flow of sediment-laden sheet flow runoff from a disturbed area.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- The fence should be designed to withstand the runoff from a 10-year peak storm event.
- All silt fences should be installed along the contour, never up or down a slope.
- On slopes with grades greater than 7%, the silt fence should be located at least 5 to 7 feet beyond the base.
- They are appropriate in areas where runoff will be occurring as low-level shallow flow, not exceeding 0.5 cfs. Silt fence is not to be used in areas with concentrated flows.
- The drainage area for silt fences generally should not exceed 0.25 acre per 100-foot fence length.
- Slope length above the fence should not exceed 100 feet.

ADVANTAGES / BENEFITS:

- Minimize impacts to receiving surface and ground waters.

DISADVANTAGES / LIMITATIONS:

- May fail based on fabric.
- Wind and sunshine can degrade material quickly.

MAINTENANCE REQUIREMENTS:

- Inspect fences regularly.
- Replace torn or damaged fabric immediately.
- Remove accumulated sediment.
- Repair slumping or undercut sections.

POLLUTANT REMOVAL

70%	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
M	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

Filter Berms

Construction BMP

Description: A gravel or stone filter berm is a temporary ridge made up of loose gravel, stone, or crushed rock. It slows and filters flow and diverts it from an open traffic area. It acts as an efficient form of sediment control. One type of filter berm is the continuous berm, a geosynthetic fabric berm that captures sand, rock, and soil.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Most suitable for road construction areas where traffic needs to be rerouted, or in traffic areas within a construction site.
- Use well-graded gravel or crushed rock to build the berm, with rock size ranging from 3/4 inches to 3 inches in diameter.
- Space berms according to the steepness of the slope. Use closer spacing as the slope increases.
- Remove and dispose of sediment that builds up, and replace the filter material.

ADVANTAGES / BENEFITS:

- The effectiveness of a rock filter berm depends on rock size, slope, soil, hydrologic, hydraulic, topographic, and sediment characteristics.
- Effectiveness has been rated at up to 95 percent for sediment removal.
- Construction materials (mainly gravel) are relatively low in cost.

DISADVANTAGES / LIMITATIONS:

- Berms are intended to be used only in gently sloping areas (less than 10 percent).
- Installing a berm and regularly cleaning and maintaining it can result in substantial labor costs. Costs are lower in areas of less traffic, gentler slopes, and low rainfall.

MAINTENANCE REQUIREMENTS:

- Inspect after every rainfall to make sure sediment has not built up and that vehicles have not damaged it.
- Repair any deterioration to keep the berm functioning properly.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
	Streambank Protection
x	**On-Site Flood Control**
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
M	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

28%	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Sediment Basin

Construction BMP

Description: A sediment basin is a temporary pond built on a construction site to capture runoff before it leaves the site. It allows a shallow pool to form in an excavated or natural depression where sediment from stormwater runoff can settle. Sediment basins are often constructed in locations that will later be modified to serve as post-construction stormwater basins.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Applicable to drainage areas greater than 2 acres.
- Provide a storage volume of at least 3,600 cf/acre of drainage area.
- Basin should drain within 72 hours.

ADVANTAGES / BENEFITS:

- Used for larger projects 5 acres or more.
- Can serve for up to 3 years with regular maintenance.

DISADVANTAGES / LIMITATIONS:

- Should not be used in areas of continuously running water.
- Do not use in areas where failure to contain stormwater will result in loss of life, or damage to homes or buildings, of prevent use of public roads or utilities.
- Standing water may cause mosquitoes or other pests to breed.
- May require large surface area to allow sediment to settle.

MAINTENANCE REQUIREMENTS:

- Should be inspected after each storm event, and any damage should be repaired immediately.
- Sediment should be removed from the basin when storage capacity has reached approximately 50 percent.
- During removal, sediment should not enter adjacent streams or drainage ways. Trash and debris from around dewatering devices should be removed promptly.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [ ] **Streambank Protection**
- [x] **On-Site Flood Control**
- [x] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
M	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

POLLUTANT REMOVAL

70%	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen removal
No data	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E. Coli removal

Sediment Trap

Construction BMP

Description: Sediment traps are small impoundments that allow sediment to settle out of runoff water. Sediment traps are typically installed in a drainage way or other point of concentrated discharge from a disturbed area. They can be constructed of rock, straw bales, or a combination of materials.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Sediment traps are easily adaptable to many conditions, including thin soils and steep slopes.
- A large contributing area may require large or multiple sediment traps.

ADVANTAGES / BENEFITS:

- Reduction of large particle sediment in surface waters.

DISADVANTAGES / LIMITATIONS:

- Not as effective as other erosion control techniques in removing small particle sediments.
- Does not provide filtration of non-sediment pollutants.

MAINTENANCE REQUIREMENTS:

- Regular maintenance includes outlet checking and sediment removal.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
x	**Streambank Protection**
	On-Site Flood Control
x	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
M	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

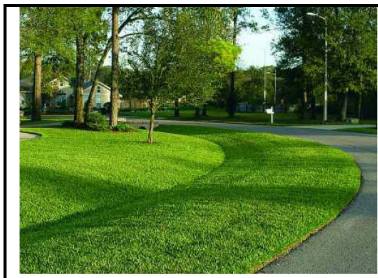
L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Diversion Swale/Berm

Construction BMP



Description: A diversion swale or berm is a channel of compacted soil constructed above, across, or below a slope, with a supporting earthen ridge on the lower side. It is designed to divert runoff from undisturbed areas upstream from a construction site to a desired outfall location.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- The top of the ridge should be at least 2 feet wide.
- Bottom width at ground level is typically 6 feet.
- The minimum height for earthen dikes is 18 inches, with side slopes no steeper than 2:1.
- The maximum design flow velocity should range from 1.5 to 5.0 feet per second, depending on the vegetative cover and soil texture.
- If the expected life span of the diversion structure is greater than 15 days, it is strongly recommended that both the earthen dike and the accompanying ditch be seeded with vegetation immediately after construction.

ADVANTAGES / BENEFITS:

- Earthen diversions are effective as temporary devices of controlling the velocity and direction of stormwater runoff.

DISADVANTAGES / LIMITATIONS:

- Swales and berms could become barriers to construction equipment.
- Regrading the site to remove the dike or swale may add additional cost.
- The concentrated runoff in the channel or ditch has increased erosion potential.

MAINTENANCE REQUIREMENTS:

- Inspect diversions regularly for damage.
- Maintain the dike at original height.
- Compact earth diversions at all times.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
L	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen removal
No data	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E. Coli removal

Temporary Slope Drain

Construction BMP

Description: Temporary slope drains are flexible or rigid conduits that extend from the top to the bottom of a cut or fill slope where there is high potential for erosion. Stormwater is routed down the slope through the pipe to a stabilized outlet, avoiding erosion of a bare slope.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Design an effective collection system to direct flow to the pipe.
- Pipe must be properly sized and anchored to the slope.
- Careful installation is important; failed slope drains often result in gully erosion on the slope and sedimentation at the slope base.
- Outlet protection must be provided.

ADVANTAGES / BENEFITS:

- Slope drains may be permanent or temporary.

DISADVANTAGES / LIMITATIONS:

- May cause gully or erosion.

MAINTENANCE REQUIREMENTS:

- Inspect storm drain after storm event to ensure slope drain is working properly.
- Inspect pipe anchors to ensure pipe is secure.
- Remove debris and sediment from inlet and outlet of pipe.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [x] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
M	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Erosion Control Blankets

Construction BMP

Description: For applications where natural vegetation alone will provide permanent erosion protection, temporary products such as netting, open weave textiles and blankets made of biodegradable materials (e.g., straw, coconut fiber) can be used. These fabrics are designed to reduce erosion from rainfall impact, hold soil in place, and absorb and hold moisture near the soil surface. It can also be used to secure mulching to the ground and stabilize soil while plants are growing.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Use when vegetation is likely to grow too slowly to provide adequate cover.
- Use when high winds render mulch an ineffective control.
- Erosion control blankets should be installed parallel to the direction of flow.
- Blanket ends should be buried at least six inches deep.
- Erosion control blankets should be placed loosely on the soil - not stretched.
- Edges should be stapled at least every three feet.

ADVANTAGES / BENEFITS

- May be used on steep slopes and in drainageways.
- Once properly installed, they can prevent migration of soil.

DISADVANTAGES / LIMITATIONS:

- Geotextiles have the potential of being sensitive to light and must be protected prior to installation.
- Some types might promote increased runoff.
- They are not suitable for excessively rocky sites.
- Not suitable for high velocity flows.

MAINTENANCE REQUIREMENTS:

- Trapped sediment should be removed after each storm event.
- All blankets and mats should be inspected periodically after installation.
- Inspect after rainstorms.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
x	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
H	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*
High Density/Ultra-Urban: *Yes*
Drainage Area: *N/A*
Soils: *N/A*
Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Surface Roughening

Construction BMP

Description: Surface roughening involves tracking, scarifying, imprinting or tilling a disturbed area to provide temporary stabilization. Roughening may be effective where mulching is not due to high winds or steep slopes. Roughening may also be the BMP of choice when activities will occur in the area within a few days. Depending on the technique used, surface roughening may also help to create conditions favorable to vegetation establishment.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Create grooves 2 to 6 inches deep and approximately 6 inches apart perpendicular to slope.
- Surface roughening is not a stand-alone BMP, and should be used in conjunction with other erosion and sediment controls.
- Horizontal grooves can be made using tracks from equipment treads, stair-step grading, ripping or tilling.

ADVANTAGES / BENEFITS:

- Roughening may be effective where mulching is not due to high winds or steep slopes.
- May be used when activities will occur in the area within a few days.

MAINTENANCE REQUIREMENTS:

- Roughened areas should be inspected frequently, especially after rain or wind.
- Reapply mulch or surface roughening as necessary.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [x] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Temporary Seeding and Planting

Construction BMP

Description: Temporary seeding used to establish a vegetative cover for disturbed areas that will be exposed longer than 14 days. Effective seeding includes preparation of the soil, selection of an appropriate seed mix, proper planting techniques, and protection of the seeded area with mulch or erosion control blankets.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Use on slopes of 3:1 or flatter.
- Soil type and site conditions should govern seed selection design.
- If soil is compacted, it should be disked, plowed, tilled or scarified to provide seed lodgement.
- Drill seeding is the preferred seeding method.

ADVANTAGES / BENEFITS:

- This BMP provides quick erosion protection for disturbed areas. Once the vegetation has been established, it traps sediment, promotes infiltration, and improves site appearance.
- It is a relatively inexpensive method of erosion control.

DISADVANTAGES / LIMITATIONS:

- Temporary seeding may not be appropriate in dry areas or areas without supplemental irrigation.
- Areas with continual construction traffic will not be able to maintain viable vegetative growth.
- Temporary seeding must have sufficient time to be established.
- Additional mulching may be necessary to protect seeding during the weeks before true establishment of vegetation.
- Temporary seeding is not to be used on steep slopes.
- Seeding may require fertilizer on poor quality soils.
- Temporary seeding is not appropriate for short-term inactivity (less than 14 days).

MAINTENANCE REQUIREMENTS:

- Inspect after rainfall.

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
x	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

Mulching

Construction BMP

Description: Mulching is used to stabilize disturbed areas by evenly applying straw, hay, shredded wood mulch, bark or compost and securing the mulch by crimping, tackifiers, netting or other measures. Mulch can be applied using mechanical dry application equipment or hydromulching equipment that applies a slurry.

KEY CONSIDERATIONS

DESIGN CRITERIA:
- The useful life is two to six months depending upon the material used.
- On steep slopes or in highly erodible soils, multiple treatments may be appropriate.

ADVANTAGES / BENEFITS:
- Rapid establishment of mulch or mulch combined with seeding can reduce runoff in cleared and graded areas by up to sixfold.

DISADVANTAGES / LIMITATIONS:
- Mulching may delay seed germination as the cover reduces the soil surface temperatures.
- The mulch cover is subject to erosion, and may be washed away in a large storm.
- Close inspections and maintenance are necessary to ensure effective erosion control.

MAINTENANCE REQUIREMENTS:
- Mulched or roughened areas should be inspected frequently, especially after rain or wind.
- Reapply mulch or surface roughening as necessary to restore coverage.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
x	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Soil Retention Measures

Construction BMP

Description: Soil retention measures consist of soil retention blankets, soil binders, turf reinforcement mats and other practices to hold soil in place and enhance vegetation establishment and survivability, particularly on slopes or in channels.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Soil type and location will dictate which material will be utilized.
- Need to understand products, characteristics and expected longevity.

ADVANTAGES / BENEFITS:

- Binders may be incorporated into earth work.
- Soil retaining structures are useful for very steep slopes or loose, highly erodible soils.

DISADVANTAGES / LIMITATIONS:

Structural methods

- Heavy rains or mass wasting may damage or destroy these structures.
- Some materials may be difficult to maintain over a longer term.

Soil binders

- Soil binders require a minimum curing time.
- Soil binders will generally experience spot failures during heavy rainfall events.
- The water quality impacts of soil binders are relatively unknown.

MAINTENANCE REQUIREMENTS:

Structural methods

- Inspect structures periodically.
- Repair any damage immediately.

Soil binders

- Inspect regularly.
- Reapply the selected soil binder as needed.

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
x	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L=Low **M**=Moderate **H**=High

Wind Erosion Controls

Construction BMP

Description: Wind erosion controls limit the movement of dust from disturbed surfaces and includes many different practices such as seeding and mulch, use of soil binders, and placement of rock or mulch. Materials such as wood fence, snow fence, vegetation (trees and shrubs) and straw bales may be used as barriers. Sprinkling areas with water and other dust palliatives may be used.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Wind erosion control practices can be applied to construction sites and other areas where loss of vegetation has occurred.
- Can be adapted in all areas where high winds are an environmental condition. In arid climates, vegetative controls may require irrigation.

ADVANTAGES / BENEFITS:

- Lowers sedimentation resulting from runoff.
- Controls airborne soil and other particulates, improving air quality.
- Wind erosion control along roads and highways may reduce snow removal costs and enhance driver safety.

DISADVANTAGES / LIMITATIONS:

- Excessive sprinkling may result in non-storm water discharges from site.

MAINTENANCE REQUIREMENTS:

- Trees and Shrubs: Weeding in the first years after installation will enhance tree survival.
- Periodic pruning is necessary to long term performance and appearance. Dead, damaged or diseased trees should be replaced.
- Other structures**:** Require periodic maintenance to replace damaged areas.

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
M	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

Steep Slope Terracing

Construction BMP

Description: Steep slope terracing involves grading to break up a slope by providing areas of low slope in the reverse direction to keep water from proceeding down slope at increasing volume and velocity. Terraces shorten the direct flow lengths across a vegetated, steep slope, helping to reduce development of rills and gullies.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Can be used on larger, sloping watersheds steeper than 4:1 (H:V).
- Type, number and spacing depends on slope and slope length.
- The Revised Universal Soil Loss Equation (RUSLE) is helpful in determining spacing of terraces.

ADVANTAGES / BENEFITS:

- Can be used on slopes too steep for other sediment control measures.
- Can be adapted in rural and urban settings.
- Can be used on larger, steeply sloping watersheds if adequately designed.
- Can be used in rural and agricultural settings as well as urban construction and other steep slope areas where high water velocity causes erosion.
- Erosion and sediment control through water velocity control.

MAINTENANCE REQUIREMENTS:

- Maintenance is required to repair areas weakened by high flows. Concentrated flows may break terrace design.
- Can provide long term control with adequate maintenance.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality Protection**
- [ ] **Streambank Protection**
- [x] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
M	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

Wattles

Construction BMP

Description: A wattle is a linear roll made of natural materials such as straw, coconut fiber, or other fibrous material trenched into the ground and held with a wooden stake. They are used as a sediment barrier to intercept sheet flow runoff from disturbed areas.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Install along the contour to avoid concentrating flows.
- The maximum tributary drainage area is approximately 0.25 acres.
- Maximum disturbed slope length is 150 feet.
- A tributary slope no steeper than 3:1. Longer and steeper slopes require additional measures.
- This recommendation only applies to wattles installed along the contour.

ADVANTAGES / BENEFITS:

- Can be used as perimeter control for stockpiles and the site.
- Can be used as part of inlet protection.
- Can be used as check dams in small drainage ditches. (Wattles are not intended for use in channels with high flow velocities.)
- Can be used on disturbed slopes to shorten flow lengths (as an erosion control device).
- Can be used as part of multi-layered perimeter control along a receiving water such as a stream, pond or wetland.
- Wattles work well in combination with other layers of erosion and sediment controls.

MAINTENANCE REQUIREMENTS:

- Wattles will eventually degrade.
- Remove accumulated sediment before the depth is one-half the height of the wattle.
- Replace damaged wattle by replacing the damaged section.

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

Concrete Washout

Construction BMP

Description: Concrete washouts are designated and managed structures on a construction site used to contain concrete and liquids when the chutes of concrete mixers and hoppers of concrete pumps are rinsed out after delivery. Three basic approaches are available: excavation of a pit, providing an above ground structure, use of prefabricated haul-away containers. Surface discharges of concrete washout water are prohibited.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Used to consolidate solids for easier disposal and prevent runoff of liquids.
- Pits must be lined unless soil has sufficient buffering capacity to protect groundwater.
- Do not place within 50 feet of storm drains, open ditches, or waterbodies.
- Number of facilities is dependent on expected demand for storage capacity.
- Provide adequate signage on construction site.

ADVANTAGES / BENEFITS:

- Reduces runoff of pollutants.

MAINTENANCE REQUIREMENTS:

- Remove or cover prior to rain events.
- Inspect structure for signs of damage or weakening.
- Cleanout and dispose of material as required when washout reaches 2/3 capacity.

STORMWATER MANAGEMENT SUITABILITY

x	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *N/A*

Soils: *N/A*

Other Considerations: *N/A*

L = Low **M** = Moderate **H** = High

POLLUTANT REMOVAL

No data	**Total Suspended Solids**
No data	**Nutrients** - Total Phosphorus / Total Nitrogen
No data	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E. Coli

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Waste Management

Description

Improperly disposed waste materials are a source of stormwater pollution. Waste materials deposited on the urban landscape can be washed off by stormwater runoff and delivered to the receiving water system. Thus, all measures that help to minimize the presence of these materials on the urban landscape can improve water quality. Proper disposal of household waste and toxics can reduce toxic substances on the land and reduce their presence in stormwater reaching the receiving waters.

Application

The following principles and actions should be addressed in SWMPs:

- **Provide for waste management.** Implement management procedures and practices to prevent or reduce the exposure and transport of pollutants in stormwater from solid, liquid and sanitary wastes that will be generated at the site. Practices such as trash disposal, recycling, proper material handling, and cleanup measures can reduce the potential for stormwater runoff to pick up construction site wastes and discharge them to surface waters. Implement a comprehensive set of waste-management practices for hazardous or toxic materials, such as paints, solvents, petroleum products, pesticides, wood preservatives, acids, roofing tar, and other materials. Practices should include storage, handling, inventory, and cleanup procedures, in case of spills.

Specific practices that should be considered include:

Solid or Construction Waste

- Designate trash and bulk waste-collection areas on-site.
- Recycle materials whenever possible (e.g., paper, wood, concrete, oil).
- Segregate and provide proper disposal options for hazardous material wastes.
- Clean up litter and debris from the construction site daily.
- Locate waste-collection areas away from streets, gutters, watercourses, and storm drains. Waste-collection areas (dumpsters, and such) are often best located near construction site entrances to minimize traffic on disturbed soils. Consider secondary containment around waste collection areas to minimize the likelihood of contaminated discharges.
- Empty waste containers before they are full and overflowing.

Sanitary and Septic Waste

- Provide convenient, well-maintained, and properly located toilet facilities on-site.
- Locate toilet facilities away from storm drain inlets and waterways to prevent accidental spills and contamination of stormwater.
- Maintain clean restroom facilities and empty portable toilets regularly.
- Where possible, provide secondary containment pans under portable toilets.
- Provide tie-downs or stake-downs for portable toilets.
- Educate employees, subcontractors, and suppliers on locations of facilities.
- Treat or dispose of sanitary and septic waste in accordance with state or local regulations. Do not discharge or bury wastewater at the construction site.
- Inspect facilities for leaks. If found, repair or replace immediately.
- Special care is necessary during maintenance (pump out) to ensure that waste and/or biocide are not spilled on the ground.

Hazardous Materials and Wastes

- Develop and implement employee and subcontractor education, as needed, on hazardous and toxic waste handling, storage, disposal, and cleanup.
- Designate hazardous waste-collection areas on-site.
- Place all hazardous and toxic material wastes in secondary containment.
- Hazardous waste containers should be inspected to ensure that all containers are labeled properly and that no leaks are present.

Cost Considerations

Collection and disposal of household wastes can be expensive. Where hazardous/ toxic wastes are involved there is a need for operators to be adequately trained, analysis to be done of unknown materials, safe transport and containers, and extensive recordkeeping. There are also regulatory requirements on how wastes can be disposed. Disposal or hazardous wastes must follow the requirements outlined in the Resource Conservation and Recovery Act and associated regulations.

Advantages and Disadvantages

The water quality improvements that can result from proper disposal methods and opportunities for recycling have not been quantified.

Advantages

Major advantages of proper disposal of waste and toxic materials can include:

1. Reduction in the quantities of solids, metals, floatable materials, oxygen-consuming materials, nutrients, fecal matter, oil and toxic substances transported by stormwater to receiving waters
2. Appropriate disposal of wastes at landfills.
3. Reduced littering on the urban landscape.
4. Increased recycling and resource recovery.
5. Improved aesthetics of public areas.

Disadvantages

Some disadvantages associated with the use of this BMP include:

1. Its success and effectiveness depends on voluntary efforts and training.
2. Places an increased demand on recycling and waste management facilities.
3. Requires dedicated space and access at construction sites.

Use of Pesticides, Herbicides, and Fertilizer

Description

Pesticides, herbicides, and fertilizers are used to maintain landscaping in residential, commercial and industrial areas. These substances are usually toxic and can contaminate surface runoff if not properly used. This nonstructural BMP encourages proper use and application of pesticides, herbicides, and fertilizers.

Application

While pesticides and herbicides are toxic to aquatic life at low concentrations, fertilizers are usually only toxic at high concentrations. Fertilizers, however, are more commonly a problem because of their nutrient-enrichment effect on receiving waterbodies. An oversupply of phosphorus and nitrogen will promote alga growth that can lead to a depletion of dissolved oxygen needed for fish and other aquatic organisms.

The rate and timing of application of pesticides, herbicides, and fertilizer are important to minimize transport by surface runoff, as well as to optimize their intended purpose in landscape maintenance. Use of these chemicals in accordance with manufacturer's recommendations can prevent most of the surface water contamination being attributed to their use.

Advantages

Major advantages of the use of this BMP include:

1. Can reduce the source of pesticides, herbicides and fertilizers entering receiving waters.
2. Can help reduce the level of phosphorus and nitrogen in receiving waters, thereby positively affecting the problem of nuisance growth of algae, and eutrophication of small lakes and tributary streams.
3. Encourage the use of less toxic or substitute methods of pest and weed control that, if followed, further reduce the supply of pesticides and herbicides for contact with surface runoff.
4. Heighten the awareness and public understanding of how individual actions can add to or reduce stormwater pollution.

Disadvantages

Some disadvantages associated with the use of this BMP include:

1. Requires on-going educational activities and the distribution of information.

Illicit Discharge Controls

Source Control BMP

Description

Illicit discharges include illegal dumping and accidental spills. Pollutants entering the storm sewer systems by illegal dumping, spills, and illicit connections can contribute to public health problems.

Application

Activities that reduce the entry of pollutants into the municipal storm sewer system from physical connections to the storm drain system of sanitary sewers and floor drains, or from illicit discharges, are accomplished through regulation, inspection, testing, and education. These include controls on illegal dumping of toxic substances and petroleum products, responses to contain accidental spills, measures to locate and disconnect illicit connections of sanitary sewers to storm sewers, and measures to prevent additional illicit sanitary sewer connections in the future.

Example Applications

Illegal Dumping:

Illegal dumping occurs whenever toxic substances or other pollutants are dumped into or disposed of directly into storm drainage facilities, or onto the urban landscape. Prohibitions against such activity have been enacted by the City of Gillette.

Accidental Spill Response:

The storage, transport and disposal of hazardous and toxic substances are regulated activities under local state and federal laws. Response procedures for management of accidental spills of pollutants are practiced presently by the City. As a result, the City Fire and Hazmat departments are equipped to respond to such spills. Nevertheless, most spills have the potential to contaminate receiving waters via transport by the storm sewer system.

Advantages

Major advantages of the use of this BMP include:

1. Can reduce pollutants entering receiving waters.
2. Can reduce public health problems.

Disadvantages

Some disadvantages associated with the use of this BMP include:

1. In some cases it can be difficult to identify source of a pollutant.

Good Housekeeping

Source Control BMP

Description

Good housekeeping practices are designed to maintain a clean and orderly work environment. The most effective first steps towards preventing pollution in stormwater from work sites simply involves using good common sense to improve the facility's/construction project's basic housekeeping methods. A clean and orderly work site reduces the possibility of accidental spills caused by mishandling of chemicals and equipment and should reduce safety hazards to personnel. A well-maintained material and chemical storage area will reduce the possibility of stormwater mixing with pollutants. Some simple procedures a facility/construction site can use to promote good housekeeping are: improved operation and maintenance of machinery and processes, material storage practices, material inventory controls, routine and regular clean-up schedules, maintaining well organized work areas, signage, and educational programs for employees about all of these practices.

Application

Operation and Maintenance:

To ensure that equipment and work related processes are working well the following practices can be implemented:

1. Maintain dry and clean floors and ground surfaces by using brooms, shovels, vacuum cleaners or cleaning machines rather than wet clean-up methods.
2. Regularly pickup and dispose of garbage and waste material.
3. Make sure all equipment and related processes are working properly and preventative maintenance is kept up with on both.
4. Routinely inspect equipment and processes for leaks or conditions that could lead to discharges of chemicals or contact of stormwater with raw materials, intermediate materials, waste materials, or products used on site.
5. Ensure all spill cleanup procedures are understood by employees. Training of employees on proper clean up procedures should be implemented.
6. Designate separate areas of the site for auto parking, vehicle refueling and routine maintenance.
7. Clean up leaks, drips and other spills immediately.
8. Cover and maintain dumpsters and waste receptacles.

Material Storage Practices:

Improperly storing of material on site can lead to the release of materials and chemicals that can cause stormwater runoff pollution. Proper storage techniques include the following:

1. Provide adequate aisle space to facilitate material transfer and ease of access for inspection.
2. Store containers, drums, and bags away from direct traffic routes to prevent accidental spills.
3. Stack containers according to manufacturer's instructions to avoid damaging the containers from improper weight distribution.
4. Store containers on pallets or similar devices to prevent corrosion of containers that results from containers coming in contact with moisture on the ground.
5. Store toxic or hazardous liquids within curbed areas or secondary containers.
6. Assign responsibility of hazardous material inventory to a limited number of people that are trained to handle such materials.

Material Inventory Practices:

An up-to-date inventory kept on all materials (both hazardous and non-hazardous) present on site will help keep material costs down caused by overstocking, track how materials are stored and handled onsite, and identify which materials and activities pose the most risk to the environment. The following description provides the basic steps in completing a material inventory:

1. Identify all chemical substances present at work site. Perform a walk through of the site, review purchase orders, list all chemical substances used and obtain Material Safety Data Sheets (MSDS) for all chemicals.
2. Label all containers. Labels should provide name and type of substance, stock number, expiration date, health hazards, handling suggestions, and first aid information. This information can also be found on a MSDS.
3. Clearly mark on the hazardous materials inventory which chemicals require special handling, storage, use and disposal considerations.

Cost Considerations

This BMP is inexpensive to implement. The primary cost is staff time.

Advantages

Major advantages of the use of this BMP include:

1. An advantage of Good Housekeeping BMPs is that they are inexpensive to implement.
2. Benefits of a clean and orderly site can go beyond stormwater quality improvement. This could include a more accurate inventory of materials on site or reduction in worker injuries, for example, slips on wet surfaces.

Disadvantages

Some disadvantages associated with the use of this BMP include:

1. A disadvantage of this BMP is that, like many nonstructural BMPs, employee awareness and education is fundamental to its effectiveness.
2. Continued awareness training is necessary to ensure that positive behaviors are maintained.

Preventative Maintenance

Source Control BMP

Description

Preventative maintenance involves the regular inspection and testing of equipment and operational systems. These inspections should uncover conditions such as cracks or slow leaks which could cause failures that result in discharges of chemicals to surface waters. The purpose of the preventative maintenance program should be to prevent failures by adjustment, repair, or replacement of equipment before a major breakdown or failure can occur.

Preventative maintenance has been practiced predominantly where excessive down time is extremely costly. As a stormwater BMP, preventative maintenance should be used selectively to eliminate or minimize the spill of contaminants to receiving waters.

Preventative maintenance should be part of a general good housekeeping program designed to maintain a clean an orderly work environment. Often the most effective first step towards preventing stormwater pollution from sites simply involves good common sense to improve the facility preventative maintenance and general good housekeeping methods.

Application

Preventative maintenance procedures and activities are applicable to almost every construction site where equipment is used.

Operation and Maintenance:

The key to properly tracking a preventative maintenance program is through the continual updating of maintenance records. Records should be updated immediately after preventative maintenance, or when any repair has been performed on any item on site. An annual review of these records should be conducted to evaluate the overall effectiveness of the preventative maintenance program. Refinements to the preventative maintenance procedures and tasking should be implemented as necessary.

Maintenance activities associated with vehicle and equipment include the following:

1. Maintain clean equipment, no excessive amounts of oil and grease buildup.
2. Use drip pans or absorbents where repairs are performed outside and in potential problem areas.
3. Use appropriate facilities to perform repairs involving exchange of fluids and lubricants and lot painting.
4. Drain and crush oil filters before recycling or disposal.
5. Clean any catch basins that receive runoff from a maintenance area.
6. Do not hose down work areas or use concrete cleaning products; use mops or dry sweeping compound. Store mechanical parts and equipment under cover.
7. Drain all fluids and remove batteries from salvage vehicles and equipment.
8. Recycle or dispose of the following in the correct manner: greases, oils, antifreeze, brake fluid, all cleaning solutions, hydraulic fluid, batteries, transmission fluid, worn parts, filters and rags, or similar items.
9. Use recycled products and substitute materials with less hazardous properties where feasible.
10. Provide employee awareness training.
11. Store solvents, greases, oils, hydraulic fluids, paints, thinners and hazardous materials indoors.
12. Store used oil for recycling in self-contained labeled tanks.
13. Keep spill response information and spill cleanup materials on the site and readily available.
14. Locate used oil tanks and drums away from the nearest inlet to the storm drainage system, flowing streams and preferably indoors if possible.

General Maintenance activities associated with a construction project include:

1. Chemical applicators should be required to adhere to all regulations regarding handling, storage and application of herbicides, insecticides, fungicides and rodenticides.
2. All chemicals should be handled and stored in compliance with their Material Safety Data Sheets (MSDS).
3. All chemicals should have their associated MSDS information sheets logged on the inventory maintained by the site administrator responsible for stormwater management.
4. Where possible, leave native vegetation undisturbed, and plant native vegetation in disturbed soil areas, to reduce irrigation, fertilizer and pesticide needs.
5. Sweeping of paved surfaces.
6. Proper disposal of wash water, sweepings and sediments should occur.

Advantages
Major advantages of the use of this BMP include:
1. Reduction in the amount of down time due to unanticipated equipment breakage.
2. Planned equipment down time.

Disadvantages
Some disadvantages associated with the use of this BMP include:
1. Additional costs.
2. Availability of trained preventative maintenance staff.

Spill Prevention and Response

Source Control BMP

Description

Spills and leaks together are one of the largest sources of stormwater pollutants, and in most cases are avoidable. The primary objective of following a spill prevention and response BMP is the prevention and reduction of discharges of pollutants to stormwater as a result of spilled products and materials.

Application

Spill Prevention Measures:

For construction project operations the following preventative strategies are:

1. Identify all equipment that may be exposed to stormwater, pollutants that may be generated and possible sources of leaks or discharges.
2. Perform regular maintenance of each piece of equipment to check for: proper operation, leaks, malfunctions, and evidence of leaks or discharge (stains). Develop a procedure for spill reporting, clean up, and repair.
3. Drain or replace motor oil or other automotive fluids in an area away from streams or storm or sanitary sewer inlets. Collect spent fluids and recycle or dispose of properly.
4. In fueling areas, clean up spills with dry clean up methods (absorbents), and use damp cloths on gas pumps and damp mops on floors instead of a hose.

An important part of spill prevention is employee training. Make sure employees are trained in spill prevention practices and adhere to them. The best way to prevent pollutants from entering the storm drains is to prevent stormwater from contacting equipment or surfaces that may have oil, grease, or other pollutants. Some good activities to help prevent negative impacts on stormwater quality include:

1. Dispose of stormwater that has collected in containment areas properly (may need permit if contaminated).
2. Adopt effective housekeeping practices.
3. Ensure adequate security.

Identification of Spill Areas:

It is important to identify potential spill areas and their drainage points to determine preventative measures and spill response actions. Areas and activities that are most vulnerable to spills include:

1. Loading and Unloading areas
2. Storage Areas
3. Dust or Particulate Generating Processes
4. Waste Disposal Activities

In addition to these areas, evaluate spill potential in other areas (access roads, parking lots, power generating facilities, etc.). It is also important to estimate the possible spill volume and drainage paths.

Spill Response Procedures and Equipment:

The recommended immediate response actions are:

1. Wipe up small spills with a shop rag, store shop rags in covered rag container, and dispose of properly (or take to professional cleaning service and inform them of the materials on the rag).
2. Contain medium-sized spills with absorbents (kitty litter, sawdust, etc.) and use inflatable berms or absorbent “snakes” as temporary booms for the spill. Store and dispose of absorbents properly. Wet/dry vacuums may also be used, but not for volatile fluids.
3. For large spills, first contain the spill and plug storm drain inlets where the liquid may migrate off-site, then clean up the spill.

Spill Plan Development:

A spill prevention and control plan may need to be developed and implemented for certain products that are stored, processed and refined. A Spill Prevention Plan identifies areas where spills can occur onsite, specifies materials handling procedures, storage requirements, and identifies spill cleanup procedures. The purpose of this plan is to establish standard operating procedures, and the necessary employee training to minimize the likelihood of accidental releases of pollutants that can contaminate stormwater runoff. Spill prevention is prudent from both an economic as well as environmental standpoint because spills increase operating costs and lower productivity. Note that this element is required as part of the Stormwater Pollution Prevention Plan (SWPPP), Operational Controls Section discussed in additional detail below. Stormwater contamination assessment, flow diversion, record keeping, internal reporting, employee training, and preventative maintenance are associated BMPs that should be incorporated into a comprehensive Spill Prevention Plan. A Spill Prevention Plan is applicable to facilities that transport, transfer, and store hazardous materials, petroleum products, and fertilizers that can contaminate stormwater runoff. An important factor of an effective spill prevention plan is quick notification of the appropriate emergency response teams. In some plants, each area or process may have a separate team leader and team experts.

Emergency spill cleanup plans should include the following information:

1. A description of the facility including the nature of the facility activity, and general types and quantities of chemicals stored at the facility.
2. A site plan showing the location of storage areas of chemicals, the location of storm drains, site drainage patterns, fire-fighting equipment and water source locations, and the location and description of any devices used to contain spills such as positive control valves.
3. Notification procedures to be implemented in the event of a spill such as phone numbers of key personnel and appropriate regulatory agencies.
4. Instructions regarding cleanup procedures.
5. Designated personnel with overall spill response cleanup responsibility.

A summary of the plan should be written and posted at appropriate points on site, identifying the spill cleanup coordinators, location of cleanup kits, and phone numbers of regulatory agencies to be contacted in the event of a spill. Cleanup of spills should begin immediately. No emulsifier or dispersant should be used.
In fueling areas, absorbent should be packaged in small bags for easy use and small drums should be available for storage of absorbent and/or used absorbent. Absorbent materials shall not be washed down the floor drain or into the storm sewer. Emergency spill containment and cleanup kits should be located at the site. The contents of the kit should be appropriate to the type and quantities of chemicals or goods stored at the facility. The following procedures should be followed when implementing an emergency spill cleanup plan:

1. Key personnel should receive formal training in plan execution with additional training to the people who are likely to be the first on the site. All employees should have a basic knowledge of spill control procedures.
2. A plan summary should be posted at appropriate site locations. The summary should include the identification of the spill cleanup coordinators, location of cleanup equipment, and phone numbers of site personnel and regulatory agencies to be contacted in the event of a spill.
3. Perform the following notifications in the event of a spill:
4. Fire Department
5. Wyoming Department of Environmental Quality
6. State Office of Emergency Services
7. National Response Center (only if spill exceeds the reportable quantity)
8. City Designated Stormwater Coordinator
9. Containment and cleanup of any spills should begin immediately.
10. Absorbents should be readily used in fueling areas.
11. An inventory of cleanup materials should be maintained onsite and strategically deployed based on the type and quantities of chemicals present.

Advantages and Disadvantages

Table 11-4 lists the advantages and disadvantages of different BMPs for spills.

Spill prevention planning can be limited by the following:

1. Lack of employee motivation to implement plan.
2. Lack of commitment from senior management.
3. Key individuals identified in the Spill Prevention Plan may not be properly trained in the areas of spill prevention, response, and cleanup.

Table 11-4
Advantages and Disadvantages of BMPs for Spill Prevention and Response

Best Management Practice	Advantages	Disadvantages
Drip pans – pans used to contain small volumes of leaks	Inexpensive; simple installation and operation; possible reuse/recycle of material; empty/ discarded containers can be used as drip pans	Small volumes; inspected and cleaned frequently; must be secured during poor weather conditions, personnel must be trained in proper disposal methods
Covering –enclosure of outdoor materials, equipment, containers, or processes	Simple and effective; usually inexpensive	Frequent inspection, possible health/ safety problems if built over certain activities, large structures can be expensive
Vehicle positioning – locating trucks or rail cars to prevent spills during transfer of materials	Inexpensive, easy, effective	May require redesign of loading and unloading areas, requires signage to designated areas
Loading/Unloading by Air Pressure or Vacuum – for transfer of dry chemicals or solids	Quick and simple; economical if materials can be recovered; minimize exposure of pollutants to stormwater	Costly to install and maintain; may be inappropriate for denser materials, site-specific design; dust collectors may need permit under Clean Air Act
Sweeping – with brooms to remove small quantities of dry chemicals/ solids exposed to precipitation	Inexpensive, no special training; recycling opportunities	Labor-intensive; limited to small releases of dry materials, requires disposal to solid waste container
Shoveling – for removal of large quantities of dry materials, wet solids and sludge	Inexpensive; recycling opportunities, remediate larger releases; wet and dry releases	Labor-intensive; not appropriate for large spills, requires backfill of excavated areas to maintain grade
Excavation – by plow or backhoe for large releases of dry material and contaminated areas	Cost effective for cleaning up dry materials release; common and simple	Less precise, less recycling and reuse opportunities, may require imported material for backfill
Dust Control (Industrial) – water spraying, negative pressure systems, collector systems, filter systems, street sweeping	May reduce respiratory problems in employees around the site; may cause less loss of material and save money; efficient collection of larger dust particles	More expensive than manual systems; difficult to maintain by plant personnel; labor and equipment intensive; street sweepers may not be effective for all pollutants

Stockpile Management

Source Control BMP

Description

These practices are implemented to reduce erosion and sediment transport and associated stormwater pollutants from typical soil, concrete, asphalt, or aggregate stockpiles found at construction sites. This shall include both areas where active and non-active stockpiles of construction materials are stored.

Application

Utilize erosion control measures as well as locate stockpiles in areas protected from areas of concentrated stormwater flow is anticipated.

Stockpile Management:

1. Stockpiles must be protected continuously and located away from areas where concentrated stormwater flow is anticipated, major drainage ways, and stormwater inlets. Stockpiles shall be covered and/or protected with a temporary perimeter sediment barrier. Stockpiles of “cold mix” asphalt shall be placed on and covered with durable plastic or comparable material at all times when not in use.
2. Temporary perimeter sediment barrier such as berms, dikes, silt fences, or sandbags must be constructed to protect stockpiles from runoff.
3. Implement wind erosion control practices as appropriate on all stockpiles
4. Waste stockpiles of concrete, solid, sanitary/septic materials, liquids, hazardous materials, and contaminated soils, shall be in accordance to Waste Management BMPs.
5. Stock piles shall not exceed ten (10) feet in height.

Advantages

Major advantages of the use of this BMP include:

1. Reduction in the amount of pollutants in storm drains and watercourses.
2. Reduction in loss of materials.

Disadvantages

Some disadvantages associated with the use of this BMP include:

1. Additional costs due to implementing erosion control practices.
2. Training of staff required to implement effective stockpile management.

SECTION TWELVE
STORMWATER QUALITY MANAGEMENT

12.1 INTRODUCTION

The character of the urban landscape affects both the quantity and the quality of stormwater discharged to receiving waters during each storm. The quality of stormwater runoff from developed lands and urbanized areas can be impacted by some or all of the sources and contaminants as indicated in Table 12-1. Increasing impermeable areas such as rooftops, parking lots and paved surfaces impacts stormwater quality by decreasing the opportunity for stormwater to infiltrate and percolate into the ground. In addition, the absence of natural surfaces and vegetation increases the volume and velocity of stormwater runoff and pollutants.

Table 12-1
Potential Sources of Stormwater Pollutants

Source	Contaminant
Vehicles, Machinery and Industry	Metals, Lubricants, Solvents, Paints
Lawn Care, Gardening	Pesticides, Herbicides, Fertilizers
Household Chemicals Cleaners, Chlorine	Paints, Solvents, Detergents, Disinfectants
Pets and Animals	Fecal Material, Organic Wastes
Parking Lots	Oil, Grease, Automotive Fluids, Sediments

This section presents the requirements for the implementation and use of permanent BMPs for long-term stormwater quality control and enhancement within the City of Gillette. Compliance with this section does not require water quality monitoring, or quantitative estimates of pollutant load removal. However, the use of a performance-based approach whereby the principles and objectives of stormwater pollutant control are addressed and applied is recommended. Permanent BMPs must be selected and implemented to best fit the conditions and requirements of each site.

12.2 CRITERIA FOR PERMANENT STORMWATER QUALITY

The City encourages all development and redevelopment projects located within City of Gillette to provide specific BMPs to enhance stormwater runoff quality from the fully developed project site. The minimum technical requirements for permanent BMP design are provided in Section 12.4 of these Drainage Criteria and further described in the Urban Drainage and Flood Control District's (UDFCD) Urban Storm Drainage Criteria Manual (USDCM) Volume 3, "Best Management Practices". The design forms provided in the USDCM Volume 3 shall be used to design permanent BMPs. The USDCM is available for download at this website: http://udfcd.org/downloads/down_critmanual_volIII.htm

When the use of permanent BMPs for developing and redeveloping sites within the City and the planning boundary is proposed, the use of a "treatment train" that controls pollutants at their sources, reduces runoff volumes and treats pollutants in runoff is recommended (UDFCD 2010).

Alternative proposals for stormwater quality enhancement may be accepted if it can be demonstrated that the proposed facility meets or exceeds treatment standards for the water quality capture volume (WQCV) for similarly applicable BMPs included in the USDCM, Volume 3. The City will make the final decision on whether an alternative BMP can be implemented for any particular site.

12.3 BASIC DESIGN PRINCIPLES AND PROCEDURES

The seven guiding principles for integrating stormwater quality measures into the overall design of new development or redevelopment are:

1. Consider stormwater quality needs early in the design process. When included in the initial planning for a project, opportunities to integrate stormwater quality facilities into a site can be fully realized.
2. Take advantage of the entire site and site topography when planning for stormwater quality treatment. Spreading runoff over a larger portion of the site can help to reduce undesirable treatment strategies that rely on proprietary underground treatment devices or deep, walled-in basins that detract from a site and are difficult to maintain.
3. Reduce runoff rates and volumes to the maximum extent practicable to more closely match natural conditions. To achieve this, place stormwater in contact with the landscape, minimize directly connected impervious areas, reduce the amount of impervious area (e.g., replace low-use or emergency access paved areas with porous pavement) and select treatment techniques that promote infiltration.
4. Integrate stormwater quality management and flood control. For example, in cases where an extended detention basin or retention pond is used to address stormwater quality, these basins can be modified to include flood control detention in addition to the WQCV if space constraints allow.
5. Design attractive stormwater quality facilities that enhance the site, the community, and the environment. Designers should consider surrounding land use, context, and the prominence of the stormwater quality facility's location and proximity of the site to important civic spaces.
6. Design facilities that can be easily maintained. Facility design should provide adequate maintenance access with a minimum disturbance, disruption, and cost.
7. Design facilities with public safety in mind. Consider minimizing perimeter wall heights, providing railing adjacent to vertical drops of 30" or more, and ensuring basin edges are designed with gradually sloping banks. Avoid walled in or steeply sloped, remote basins that are not easily accessed.

12.3.1 BMP Selection

Selection of BMPs involves factors such as physical site characteristics, treatment objectives, aesthetics, safety, and cost. Typically, there is not a single answer to the question of which BMP (or BMPs) should be selected for a site—there are usually multiple solutions ranging from stand-alone BMPs to treatment trains that combine multiple BMPs to achieve the water quality objectives. Understanding the physical site and space constraints, treatment objectives and other factors will assist the designer in the proper selection of permanent BMPs for the project and facilitate the completion of the engineering calculations and specifications to support the BMP design.

Physical and space constraints are factors that should be considered when selecting BMPs, as discussed in the following paragraphs.

12.3.1.1 Physical Constraints

The first step in selecting permanent BMPs is identification of physical characteristics of a site including topography, soils, contributing drainage area, groundwater, baseflows, wetlands, existing drainageways, development conditions in the tributary watershed (e.g., construction activity), and other factors. A fundamental concept of Low Impact Development (LID) is preservation and protection of site features such as wetlands, drainageways, tree canopy, and soils that are conducive to infiltration and evaporation, which provide water quality and other benefits. LID stormwater treatment systems are designed to take advantage of these natural resources. For example, if a portion of a site is known to have soils with high permeability, this area may be well-suited for bioretention or a permeable pavement system. Areas of existing wetlands, which may be difficult to develop from a Section 404 permitting perspective, could be considered for polishing of runoff following BMP treatment, providing additional water quality treatment for the site while at the same time enhancing the existing wetlands with additional water supply in the form of treated runoff. See Section 12.6 for additional discussion regarding LID.

Some physical site characteristics that provide opportunities for BMPs or constrain BMP selection include:

- **Contributing drainage area—**Contributing drainage area is an important consideration both on the site level and at the regional level. On the site level, there is a practical minimum size for certain BMPs, largely related to the ability to drain the WQCV over the required drain time. For example, it is technically possible to size the WQCV for an extended detention basin for a half-acre site; however, designing a functional outlet to release the WQCV over a 40-hour drain time is practically impossible due to the very small orifices that would be required. In such cases, a BMP that infiltrates the WQCV or releases it via an underdrain would be more appropriate. At the other end of the spectrum, there must be a limit on the maximum drainage area for a regional facility to assure adequate treatment of rainfall events that may produce runoff from only a portion of the area draining to the BMP. If the overall drainage area is too large, events that produce runoff from only a portion of the contributing area will pass through the BMP outlet (sized for the full drainage area) without adequate residence time in the BMP. In addition to size, the development status of the tributary watershed is also important. For example, infiltration-based BMPs are not allowed where the upstream watershed is under construction due to the potential for clogging of infiltration media.

- **Soils—**Soils with good permeability, most typically associated with Hydrologic Soil Groups (HSGs) soils A and B, provide opportunities for infiltration of runoff and are well-suited for infiltration-based BMPs such as bioretention, permeable pavement systems, sand filter basins, swales and buffers, often without the need for an underdrain system. Even when soil permeability is low, these types of BMPs may be feasible if soils are amended to increase permeability or if an underdrain system is used. In some cases, however, soils restrict the use of infiltration-based BMPs. When soils with moderate to high swell potential are present, infiltration should be avoided to minimize damage to adjacent structures due to water-induced swelling. In some cases, infiltration-based designs can still be used if an impermeable liner and underdrain system are included in the design. However, when the risk of damage to adjacent infrastructure is high, infiltration-based BMPs are not appropriate. The designer should consult with a geotechnical engineer for evaluating the suitability of soils for different BMP types and establishing minimum distances between infiltration BMPs and structures.

- **Groundwater—**Shallow groundwater on a site presents challenges for BMPs that rely on infiltration and for BMPs that are intended to be dry between storm events. Shallow groundwater may limit the ability to infiltrate runoff or result in unwanted groundwater storage in areas intended for storage of the WQCV (e.g., porous sub-base of a permeable pavement system or in the bottom of an otherwise dry facility such as an extended detention basin). Conversely, for some types of BMPs such as wetland channels or constructed wetland basins, groundwater can be beneficial by providing saturation of the root zone and/or a source of baseflow. Groundwater quality protection is an issue that should be considered for infiltration-based BMPs. Infiltration BMPs may not be appropriate for land uses that involve storage or use of materials that have the potential to contaminate groundwater underlying a site (e.g., "hot spot" runoff from fueling stations or materials storage areas). If groundwater or soil contamination exists on a site and it will not be remediated or removed as a part of construction, it may be necessary to avoid infiltration-based BMPs or use a durable liner to prevent infiltration into contaminated areas.

- **Baseflows—**Baseflows are necessary for the success of some BMPs such as constructed wetland basins, retention ponds and wetland channels. Without baseflows, these BMPs will become dry and unable to support wetland vegetation. For BMPs that rely on wetland vegetation or that are intended to have a permanent pool of water, a hydrologic budget should be evaluated. Always check to see that there are no water rights requirements that may not allow for the construction of a permanent BMP.

- **Watershed development activities—**The degree to which construction in a watershed is ongoing or planned in the future is also an important factor in BMP selection. For developing watersheds, additional protection may be required for BMPs that are susceptible to clogging from sediment. For highly developed watersheds, BMP selection may be driven by space constraints or the potential to retrofit existing drainage features. The designer should consider existing, interim & future land use to select the most appropriate BMPs.

12.3.1.2 Space Constraints

Space constraints are frequently cited as a feasibility issue for BMPs, especially for high-density, lot-line-to-lot-line development and redevelopment sites. In some cases, constraints due to space limitations arise because adequate spaces for BMPs are not considered early enough in the planning process. This is most common when a site plan for roads, structures, and lots is developed and BMPs are squeezed into the remaining spaces. The most effective and integrated BMP designs begin by determining areas of a site that are best suited for BMPs (e.g., natural low areas, areas with well-drained soils) and then designing the layout of roads, buildings, and other site features around the existing drainage and water quality resources of the site.

Water quality BMPs can occupy up to 7 percent of a site's total area, exclusive of flood control detention storage requirements. Allocating this amount of land to water quality infrastructure during early planning stages will result in better integration of water quality facilities with other site features.

Although a variety of BMPs may improve water quality at a given development site, the City requires that additional factors related to the development type, aesthetics, surrounding land use, long-term sustainability and maintenance be taken into account in selection and implementation of stormwater quality BMPs. Table 12-2 summarizes typical land uses/development types present in Gillette, and Table 12-3 identifies BMPs appropriate for use in these settings.

Table 12-2
Development Type Summary

Development Type	Percentage Landscape	Percentage Parking/Paving	Building Footprint	Parking
Urban	0-5%	0-5%	90-100%	Structure
High Density Mixed Use	0-10%	0-15%	80-90%	Surface structure
Campus	15-30%	10-25%	45-75%	Surface structure
Industrial	10-15%	40-60%	25-50%	Surface
Low Density Mixed Use	10-25%	30-50%	25-60%	Surface
Residential	40-70%	5-20%	10-45%	Surface
Parks and Natural Areas	80-95%	5-15%	0-10%	Surface

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Table 12-3
BMP Application

Development Type	Runoff Reduction		Stormwater Quality Detention			Possible Flood Control Detention		
	Porous Pavement	Grass Buffers and Swales	Porous Landscape Detention	Porous Pavement Detention	Dry Ponds: Extended Detention and Sand Filter Basins	Wet Ponds: Constructed Wetland Basin and Retention Ponds	Landscape Areas	Parking Lots
Urban			X	X				
High Density Mixed Use			X	X				
Campus	X	X	X	X	X		X	X
Industrial					X		X	X
Low Density Mixed Use	X	X	X	X	X		X	X
Residential	X	X	X	X	X		X	
Parks and Natural Areas Open Space	X	X	X	X		X	X	X

Highly Applicable	X
Somewhat Applicable	
Not Recommended	

Notes:

1. Porous pavement and porous pavement detention may be used in parking areas and other low-use areas where there is no likelihood of groundwater contamination.
2. Porous landscape detention may be applied in the vicinity of buildings, in parking lot islands, and in other landscape areas where there is no likelihood of groundwater contamination or geotechnical concerns. Wherever porous landscape detention is used, geotechnical issues related to building foundation drainage and expansive soils must be addressed.
3. To avoid constrained configurations of forebays, low-flow channels, and outlet structures, extended detention basins are generally recommended only for drainage areas exceeding 1.0 acre, although sand filter detention basins may be used for areas less than 1.0 acre. Sand-filter detention basins maybe considered for use in Ultra Urban and High Density Mixed Use land uses.
4. Constructed wetland basins and retention ponds may generally be used only for drainage areas exceeding 1.0 acre that have sufficient base flow to support wetlands and permanent pools, water rights considerations need to be addressed.

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12.3.2 Design Approach

The following is a design approach for implementing the seven principles of permanent BMP selection and design:

1. Develop an initial site design. This should include a rough layout of lots, buildings, streets, parking, and landscape areas with a general idea of proposed site grades and an estimate of approximate areas associated with roofs, streets, walks, parking lots, and landscaping or open space.
2. Consider the full range of BMP alternatives. Determine which of the seven development types in Table 12-3 most closely match the site and then consider the full range of alternative approaches for addressing drainage and stormwater quality for the site, including techniques to reduce runoff and distribute BMPs throughout the site. Reduce runoff volume to the maximum extent practicable by implementing practices that minimize directly connected impervious area and promote infiltration. Test the influence of several alternatives on the overall character and layout of the site, weigh pros and cons of each, and progress towards an optimum approach. Consider long-term or life-cycle costs in the selection of alternative BMPs. When selecting and designing BMPs that rely on infiltration (i.e., porous pavement detention, porous landscape detention), carefully consider geotechnical and foundation issues and the ability of the property owner to understand and properly maintain these facilities.
3. Pursue a functional distribution of landscape areas to:
    - Keep detention basins shallow and provide some space for tree and shrub plantings around the perimeter that do not restrict maintenance access.
    - Reserve an initial area about 5 to 15 percent of the size of the impervious area for stormwater quality treatment, which may be reduced in later stages of design.
    - Minimize exclusive reliance on extended detention basins (primarily for aesthetic and land use reasons). When included, locate them near a low-lying area of the site away from pedestrian corridors and gathering places.
    - Distribute landscaped areas/grass buffers, porous landscape and porous pavement areas throughout the site. In general, it is prudent to locate porous landscape detention in close proximity to the impervious area being served.
4. Consider surface conveyance as an alternative to pipes. Conveying flows on the surface is the best method for getting runoff to porous landscape and porous pavement detention because it allows the facilities to be shallow. If flow can be conveyed on the surface in grass swales or in strips of modular block porous pavement, additional stormwater quality benefits will accrue and the required WQCV will be reduced. If runoff must be conveyed under the surface in a pipe, area inlets within a landscaped area are preferred over street or curb inlets, since this gives runoff a chance to sheet flow through vegetation and infiltrate prior to entering the storm drain. The basin or channel receiving these flows must be deep enough to allow the opposite end of the pipe to empty.

5. Integrate stormwater quality and flood control detention. Identify flood control detention requirements, water quality treatment requirements and opportunities to integrate these functions into multi-purpose facilities.
6. Tailor approach to the specific pollutants of concern. If downstream receiving waters are threatened by specific stormwater constituents, such as lakes threatened by excessive phosphorus loading leading to eutrophication (e.g., Fishing Lake), provide BMPs that are particularly effective at addressing that pollutant.

12.3.3 Targeted Pollutants and BMP Processes

BMPs have the ability to remove pollutants from runoff through a variety of physical, chemical and biological processes. The processes associated with a BMP dictate which pollutants the BMP will be effective at controlling. Primary processes include peak attenuation, sedimentation, filtration, straining, adsorption/absorption, biological uptake and hydrologic processes including infiltration and evapotranspiration. For many sites, a primary goal of BMPs is to remove gross solids, suspended sediment and associated particulate fractions of pollutants from runoff. Processes including straining, sedimentation, and infiltration/filtration are effective for addressing these pollutants. When dissolved pollutants are targeted other processes including adsorption/absorption and biological uptake are necessary. These processes are generally sensitive to media composition and contact time, oxidation/reduction potential, pH and other factors. In addition to pollutant removal capabilities, many BMPs offer channel stability benefits in the form of reduced runoff volume and/or reduced peak flow rates and extended releases for frequently occurring events. Brief descriptions of several key processes, generally categorized according to hydrologic and pollutant removal functions include:

12.3.3.1 Hydrologic Processes

- **Flow Attenuation**: BMPs that capture and slowly release the WQCV help to reduce peak discharges. In addition to slowing runoff, volume reduction may also be provided to varying extents in BMPs providing the WQCV.
- **Infiltration:** BMPs that infiltrate runoff reduce both runoff peaks and surface runoff volumes. The extent to which runoff volumes are reduced depends on a variety of factors such as whether the BMP is equipped with an underdrain and the characteristics and long-term condition of the infiltrating media. Examples of infiltrating BMPs include sand filters, bioretention and permeable pavements. Water quality treatment processes associated with infiltration can include filtration and sorption.
- **Evapotranspiration**: Runoff volumes can be reduced through the combined effects of evaporation and transpiration in vegetated BMPs. Plants extract water from soils in the root zone and transpire it to the atmosphere. Evapotranspiration is the hydrologic process provided by vegetated BMPs, whereas biological uptake may help to reduce pollutants in runoff.

12.3.3.2 Pollutant Removal/Treatment Processes

- **Sedimentation:** Gravitational separation of particulates from urban runoff, or sedimentation, is a key treatment process provided by BMPs that capture and slowly release runoff. Settling velocities are a function of characteristics such as particle size, shape and density, and fluid density and viscosity. Smaller particles under 60 microns in size (fine silts and clays) (Stahre and Urbonas 1990) can account for approximately 80 percent of the metals in stormwater attached or adsorbed along with other contaminants and can require long periods of time to settle out of suspension. Extended detention allows smaller particles to agglomerate into larger ones (Randall et al. 1982), and for some of the dissolved and liquid state pollutants to adsorb to suspended particles, thus removing a larger proportion of them through sedimentation. Sedimentation is the primary pollutant removal mechanism for many treatment BMPs including extended detention basins, retention ponds, and constructed wetland basins.

- **Straining:** Straining is physical removal or retention of particulates from runoff as it passes through a BMP. For example, grass swales and grass buffers provide straining of sediment and gross solids in runoff. Straining can be characterized as coarse filtration.

- **Filtration:** Filtration removes particles as water flows through media (often sand or engineered soils). A wide variety of physical and chemical mechanisms may occur along with filtration, depending on the filter media. Metcalf and Eddy (2003) describe processes associated with filtration as including straining, sedimentation, impaction, interception, adhesion, flocculation, chemical adsorption, physical adsorption and biological growth. Filtration is a primary treatment process provided by infiltration BMPs. Particulates are removed at the ground surface and upper soil horizon by filtration, while soluble constituents can be absorbed into the soil, at least in part, as the runoff infiltrates into the ground. Site-specific soil characteristics, such as permeability, cation exchange potential, and depth to groundwater or bedrock are important characteristics to consider for filtration (and infiltration) BMPs. Examples of filtering BMPs include sand filters, bioretention, and permeable pavements with a sand filter layer.

- **Adsorption/Absorption**: In the context of BMPs, sorption processes describe the interaction of waterborne constituents with surrounding materials (e.g., soil, water). Absorption is the incorporation of a substance in one state into another of a different state (e.g., liquids being absorbed by a solid). Adsorption is the physical adherence or bonding of ions and molecules onto the surface of another molecule. Many factors such as pH, temperature, ionic state and others affect the chemical equilibrium in BMPs and the extent to which these processes provide pollutant removal. Sorption processes often play primary roles in BMPs such as constructed wetland basins, retention ponds, and bioretention systems. Opportunities may exist to optimize performance of BMPs through the use of engineered media or chemical addition to enhance sorption processes.

- **Biological Uptake:** Biological uptake and storage processes include the assimilation of organic and inorganic constituents by plants and microbes. Plants and microbes require soluble and dissolved constituents such as nutrients and minerals for growth. These constituents are ingested or taken up from the water column or growing medium (soil) and concentrated through bacterial action, phytoplankton growth, and other biochemical processes. In some instances, plants can be harvested to remove the constituents permanently. In addition, certain biological activities can reduce toxicity of some pollutants and/or possible adverse effects on higher aquatic species. Unfortunately, not much is understood yet about how biological uptake or activity interacts with stormwater during the relatively brief periods it is in contact with the biological media in most BMPs, with the possible exception of retention ponds between storm events (Hartigan 1989). Bioretention, constructed wetlands, and retention ponds are all examples of BMPs that provide biological uptake.

When selecting BMPs, it is important to have realistic expectations of effluent pollutant concentrations. The International Stormwater BMP Database (www.bmpdatabase.org) provides BMP performance information that is updated periodically, and summarized in Table 2-2 of the USDCM Volume 3. BMPs also provide varying degrees of volume reduction benefits. Both pollutant concentration reduction and volume reduction are key components in the whole life cycle cost tool (UD-BMP Costs) (Olson et al. 2009) discussed later in the USDCM, Volume 3.

It is critical to recognize that for BMPs to function effectively, meet performance expectations and provide for public safety, BMPs must meet these three conditions:

1. Be designed according to these Drainage Criteria, taking into account site-specific conditions (e.g., high groundwater, expansive clays and long-term availability of water).
2. Be constructed as designed. This is important for all BMPs, but appears to be particularly critical for permeable pavements, bioretention and infiltration-oriented facilities.
3. Be properly maintained to function as designed. Although all BMPs require maintenance, infiltration-oriented facilities are particularly susceptible to clogging without proper maintenance. Underground facilities can be vulnerable to maintenance neglect because maintenance needs are not evident from the surface without special tools and procedures for access. Maintenance is not only essential for proper functioning, but also for aesthetic and safety reasons. Inspection of facilities is an important step to identify and plan for needed maintenance.

12.3.4 Storage-Based Versus Conveyance-Based

BMPs in these Drainage Criteria generally fall into two categories: 1) storage-based and 2) conveyance-based. Storage-based BMPs provide the WQCV and include bioretention, extended detention basins, sand filter extended detention, constructed wetland basins, retention ponds, permeable pavement systems with storage, and other similar systems. Conveyance-based BMPs generally include swales, grass buffers, constructed wetlands channels and other BMPs that provide only incidental storage. Conveyance-based BMPs are typically less land-intensive than storage-based BMPs and operate primarily by disconnecting impervious area on a site. The most effective site plans typically use a combination of conveyance based and storage-based BMPs.

12.3.4.1 Volume Reduction

BMPs that promote infiltration or that incorporate evapotranspiration have the potential to reduce the volume of runoff generated. Volume reduction is a fundamental objective of LID. Volume reduction has many benefits, both in terms of hydrology and pollution control. While stormwater regulations have traditionally focused on runoff peak flow rates, emerging stormwater regulations require BMPs to mimic the pre-development hydrologic budget to minimize effects of hydromodification. From a pollution perspective, decreased runoff volume translates to decreased pollutant loads. Volume reduction has economic benefits, including potential reductions in storage requirements for minor and major events, reduced extent and sizing of conveyance infrastructure, and cost reductions associated with addressing channel stability issues. The UDFCD has developed a computational methodology (UDFCD 2006) for quantifying volume reduction that is discussed in Chapter 3 of the USDCM, Volume 3.

Infiltration-based BMPs can be designed with or without underdrains, depending on soil permeability and other site conditions. The most substantial volume reductions are generally associated with BMPs that have permeable sub-soils and allow infiltration to deeper soil strata and eventually groundwater. For BMPs that have underdrains, there is still potential for volume reduction—as runoff infiltrates through BMP soils to the underdrain, moisture is retained by soils. The moisture eventually evaporates or is taken up by vegetation, resulting in volume reduction. Runoff that drains from these soils by gravity to the underdrain system behaves like interflow from a hydrologic perspective with a delayed response that reduces peak rates. Although the runoff collected in the underdrain system is ultimately discharged to the surface, on the time scale of a storm event, there are volume reduction benefits.

Although effects of evapotranspiration are inconsequential on the time scale of a storm event, on an annual basis, volume reduction due to evapotranspiration for vegetated BMPs can be an important component of the hydrologic budget. Between events, evapotranspiration lowers soil moisture content and permanent pool storage, providing additional storage capacity for subsequent events.

12.3.5 Pretreatment

These Drainage Criteria call for sedimentation forebays for extended detention basins, constructed wetland basins, and retention ponds. The purpose of forebays is to settle out coarse sediment and skim off floatables prior to reaching the main body of the facility. Although in many cases the forebay will be integrated with the overall treatment facility, another option to consider is a separate facility upstream. If this option is selected, the recommended size of the forebay facility is at least 20 percent of the WQCV with a design drain time of one hour. Using this approach, the size of the main WQCV facility may be reduced by 10 percent, requirements for sediment storage in the main facility may be reduced by one-half, and the forebay within the main facility may be eliminated.

It is extremely important that high sediment loading be controlled for BMPs that rely on infiltration, including permeable pavement systems, bioretention, and sand filter extended detention basins. These facilities should not be brought on-line until the end of the construction phase when the tributary drainage area has been stabilized with permanent surfaces and landscaping.

12.3.6 Treatment Train

The term "treatment train" refers to multiple BMPs in series (e.g., a disconnected roof downspout draining to a grass swale draining to a constructed wetland basin). Engineering research over the past decade has demonstrated that treatment trains are one of the most effective methods for management of stormwater quality. Advantages of treatment trains include:

- Multiple processes for pollutant removal—there is no "silver bullet" for a BMP that will address all pollutants of concern as a stand-alone practice. Treatment trains that link together complementary processes expand the range of pollutants that can be treated with a water quality system and increase the overall efficiency of the system for pollutant removal.
- Redundancy—given the natural variability of the volume, rate and quality of stormwater runoff and the variability in BMP performance, using multiple practices in a treatment train can provide more consistent treatment of runoff than a single practice and also provides redundancy in the event that one component of a treatment train is not functioning as intended.
- Maintenance—BMPs that remove trash, debris, coarse sediments and other gross solids are a common first stage of a treatment train. From a maintenance perspective, this is advantageous since this first stage creates a well-defined, relatively small area that can be cleaned out routinely. Downgradient components of the treatment train can be maintained less frequently and will benefit from reduced potential for clogging and accumulation of trash and debris.

12.3.7 On-line versus Off-line Facility Locations

The location of WQCV facilities within a development site and watershed requires thought and planning. Often, this decision-making occurs during a master planning process. The City's Stormwater Master Plan report may call for a few large regional detention facilities, smaller sub-regional facilities, or an on-site approach. Early in the development process, it is important to determine if the City's Master Plan addresses stormwater quality and then to follow the Plan's recommendations.

If the Master Plan does not identify the type and location of storm water quality facilities, a key decision involves whether to locate a BMP on-line or off-line. On-line refers to locating a BMP on a waterway that traverses a site such that all of the runoff from the upstream watershed flows through the facility. A single on-line BMP can treat both site runoff and upstream offsite runoff. Locating BMPs offline requires that all onsite catchment areas flow though a BMP prior to entering the waterway. Off-line BMPs do not provide treatment of runoff from any upstream drainage areas.

On-line WQCV facilities are only recommended if the offsite watershed has less impervious area than that of the onsite watershed. Nevertheless, on-line WQCV facilities must be sized to serve the entire upstream watershed, based on future development conditions. This recommendation is true even if upstream developments have installed their own WQCV facilities.

12.3.8 Integration with Flood Control

In addition to water quality requirements, most projects will require detention for flood control, whether on-site or in a sub-regional or regional facility. In many cases, it is efficient to combine facilities since the land requirements for a combined facility are lower than for two separate facilities. Wherever possible, it is recommended that WQCV facilities be incorporated into flood control detention facilities. This is relatively straightforward for an extended detention basin, constructed wetland pond, or a retention pond. Flood control detention could be provided above the WQCV for bioretention, provided that the facility is designed to release the detained volume in accordance with allowable release rates in the Storage chapter in these Drainage Criteria.

For sizing a combined water quality and quantity detention facility, these Drainage Criteria suggest the following approach as a minimum:

- Water Quality: The full WQCV is to be provided according to the design procedures documented in this manual.
- Minor Storm: The full WQCV plus the full minor storm quantity detention volume is to be provided.
- 100-Year Storm or Other Major Storm Event: One-half the WQCV plus the full 100-year or other major storm event detention volume is to be provided.
- Excess Urban Runoff Volume (EURV): Provide the EURV, which includes the WQCV in "full spectrum" detention designs.

The Storage chapter in Volume 2 of the USDCM provides design criteria for the full spectrum detention concept, which shows more promise in controlling the peak flow rates in receiving waterways than the multi-stage designs described above. Full spectrum detention not only addresses the WQCV for controlling water quality and runoff from frequently occurring runoff events, but extends that control for all return periods of runoff from the 2-year to the 100-year events and closely matches historic peak flows downstream. Finally, designers should also be aware that water quality BMPs, especially those that promote infiltration, can result in volume reductions for flood storage. These volume reductions are most pronounced for frequently occurring events, but even in the major event, some reduction in detention storage volume can be achieved if volume-reduction BMPs are widely used on a site. Additional discussion on volume-reduction benefits, including a methodology for quantifying effects on detention storage volumes, is provided in Chapter 3 of USDCM, Volume 3.

12.3.9 Land Use, Compatibility with Surroundings, and Safety

Stormwater quality areas can add interest and diversity to a site, serving multiple purposes in addition to providing water quality functions. Gardens, plazas, rooftops, and even parking lots can become amenities and provide visual interest while performing stormwater quality functions and reinforcing urban design goals for the neighborhood and community. Avoiding the placement of stormwater quality facilities along critical street frontage may be necessary to discourage detrimental "gaps" in the continuity of important urban spaces. The integration of BMPs and associated landforms, walls, landscape, and materials can reflect the standards and patterns of a neighborhood and help to create lively, safe, and pedestrian-oriented districts. The quality and appearance of stormwater quality facilities should reflect the surrounding land use type, the immediate context, and the proximity of the site to important civic spaces. Aesthetics will be a more critical factor in highly visible urban commercial and office areas than at a heavy industrial site.

Public access to BMPs should be considered from a safety perspective. The highest priority of engineers and public officials is to protect public health, safety, and welfare. Stormwater quality facilities must be designed and maintained in a manner that does not pose health or safety hazards to the public. As examples, steeply sloped and/or walled ponds should be avoided.

Where this is not possible, emergency egress, lighting and other safety considerations should be incorporated. Facilities should be designed to reduce the likelihood and extent of shallow standing water that can result in mosquito breeding, which can be a nuisance and a public health concern (e.g., West Nile virus). The potential for nuisances, odors and prolonged soggy conditions should be evaluated for BMPs, especially in areas with high pedestrian traffic or visibility.

12.3.10 Maintenance and Sustainability

Maintenance of permanent stormwater quality facilities should also be considered early in the planning and design phase. Even when BMPs are thoughtfully designed and properly installed, they can become eyesores, breed mosquitoes, and cease to function if not properly maintained. BMPs can be more effectively maintained when they are designed to allow easy access for inspection and maintenance and to take into consideration factors such as property ownership, easements, visibility from easily accessible points, slope, vehicle access, and other factors. For example, fully consider how and with what equipment BMPs will be maintained into the future. Clear, legally-binding written agreements assigning maintenance responsibilities and committing adequate funds for maintenance are also critical.

Sustainability of BMPs is based on a variety of considerations related to how the BMP will perform over time. For example, vegetation choices for BMPs determine the extent of supplemental irrigation required. Choosing native or drought-tolerant plants and seed mixes (as recommended by the Campbell County Conservation District) helps to minimize irrigation requirements following plant establishment. Other sustainability considerations include watershed conditions. For example, in watersheds with ongoing development, clogging of infiltration BMPs is a concern. In such cases, a decision must be made regarding either how to protect and maintain infiltration BMPs, or whether to allow use of infiltration practices under these conditions.

The City requires that BMPs are properly maintained. Maintenance guidelines are provided in the USDCM Volume 3 and are hereby incorporated by reference. For all stormwater management facilities, the City requires the following:

1. Facilities must be designed to be readily maintainable with clearly specified long-term maintenance requirements.
2. Long-term maintenance of the BMP must be provided by the facility owner.

12.3.11 Costs

Costs are a fundamental consideration for BMP selection, but often the evaluation of costs during planning and design phases of a project focuses narrowly on up-front, capital costs. A more holistic evaluation of life-cycle costs that includes consideration of operation, maintenance and rehabilitation is prudent. From a municipal perspective, cost considerations are even broader, involving costs associated with off-site infrastructure, channel stabilization and/or rehabilitation, and protection of community resources from effects of runoff from urban areas.

12.4 RECOMMENDED STRUCTURAL BMPs

The City has developed a set of recommended permanent structural BMPs listed in Table 12-4. Fact sheets are provided in Section 12.8. These fact sheets have been developed by the City as a guide for determining practical applications for a site, and provide a description of the BMP, design criteria, advantages and limitations, maintenance and costs, assuming there was material to reference for each of the sections. The stormwater management suitability section provides guidance to the designer in understanding the appropriate applications of that BMP. Depending on the amount of land required, cost, and required maintenance for each BMP, an L, M, H is placed within the box in front of each of the categories. If there is low requirement, an L is placed, moderate requirement, an M is placed and high requirement, an H is placed. The determination of L, M, H are based on the relative (qualitative) comparison with the other listed BMPs. Note that for those sites within the watershed of an impaired waterbody, additional treatment may be needed to implement measures that go beyond the minimum design criteria provided in the Fact Sheets.

In addition to the information contained in Table 12-4, the City recommends that the designer also reference the USDCM, Volume 3, which includes an extensive discussion of the development of stormwater quality controls and regionally acceptable BMP's maintenance requirements.

Erosion and sediment control BMPs for construction are discussed in Section 11 of this manual.

Table 12-4
List of Recommended Permanent BMPs

BMP	Purpose
Permanent Seeding	Runoff Reduction
Sod Stabilization	Runoff Reduction
Grass Swales	Runoff Reduction
Streambank Stabilization	Runoff Reduction
Permeable Pavements	Runoff Reduction
Stormwater Planter	Runoff Reduction
Dry Well	Runoff Reduction
Tree Box Filter	Runoff Reduction
Green Roof	Runoff Reduction
Wet Pond	Runoff Reduction
Sand Filter	Runoff Reduction/Stormwater Quality Detention
Bioretention Cells	Runoff Reduction/Stormwater Quality Detention
Infiltration Trenches	Stormwater Quality Detention
Infiltration Basins	Stormwater Quality Detention
Extended Detention Pond	Stormwater Quality Detention
Inlets/Oil-Water Separators	Stormwater Quality Detention
Retention Pond	Stormwater Quality Detention/Possible Flood Control Detention
Constructed Wetland Pond	Stormwater Quality Detention/Possible Flood Control Detention

12.5 SOURCE CONTROL BMPs

This section contains guidance on the evaluation, selection and use of source control BMPs as it applies to post-construction facilities. Table 12-5 lists the recommended source control BMPs. The Fact Sheets in Section 11.5.4 provides additional background information on source control BMPs and their importance in protecting water quality.

Table 12-5
List of Recommended Source Control BMPs for Permanent Installations

BMP	Purpose
Waste Management	Good Housekeeping
Use of Pesticides, Herbicides and Fertilizer	Good Housekeeping
Spill Prevention and Response	Good Housekeeping
Outside Material Storage (Good Housekeeping)	Good Housekeeping

12.6 GREEN INFRASTRUCTURE AND LOW IMPACT DEVELOPMENT (LID)

"Green infrastructure" is a relatively new and flexible term, and it has been used differently in different contexts. However, for the purposes of EPA's efforts to implement the Green Infrastructure Statement of Intent, EPA intends the term "green infrastructure" to generally refer to systems and practices that use or mimic natural processes to infiltrate, evapotranspirate (the return of water to the atmosphere either through evaporation or by plants), or reuse stormwater or runoff on the site where it is generated. Green infrastructure can be used at a wide range of landscape scales in place of, or in addition to, more traditional stormwater control elements to support the principles of LID. EPA is promoting green infrastructure to manage wet weather impacts in urban areas. Additional information on EPA's 2008 Action Strategy for green infrastructure can be found at http://www.epa.gov/npdes/pubs/gi_action_strategy.pdf.

At the largest scale, the preservation and restoration of natural landscape features (such as forests, floodplains and wetlands) are critical components of green infrastructure. By protecting these ecologically sensitive areas, communities can improve water quality while providing wildlife habitat and opportunities for outdoor recreation.

On a smaller scale, green infrastructure practices include rain gardens, green roofs, infiltration planters, trees and tree boxes, and rainwater harvesting for non-potable uses such as toilet flushing and landscape irrigation. See the following website for design approaches using green infrastructure:

http://cfpub.epa.gov/npdes/greeninfrastructure/technology.cfm.

LID is a site design strategy with a goal of maintaining or replicating the predevelopment hydrologic regime through the use of design techniques to create a functionally equivalent hydrologic landscape. Hydrologic functions of storage, infiltration, and ground water recharge, as well as the volume and frequency of discharges are maintained through the use of integrated and distributed micro-scale stormwater retention and detention areas, reduction of impervious surfaces, and the lengthening of flow paths and runoff time (Coffman 2000). Other strategies include the preservation/protection of environmentally sensitive site features such as riparian buffers, wetlands, steep slopes, valuable (mature) trees, floodplains, woodlands and highly permeable soils.

LID principles are based on controlling stormwater at the source by the use of microscale controls that are distributed throughout the site. This is unlike conventional approaches that typically convey and manage runoff in large facilities located at the base of drainage areas. These multifunctional site designs incorporate alternative storm water management practices such as functional landscape that act as stormwater facilities, flatter grades, depression storage and open drainage swales. This system of controls can reduce or eliminate the need for a centralized BMP facility for the control of stormwater runoff.

Although traditional stormwater control measures have been documented to effectively remove pollutants, the natural hydrology is still negatively affected (inadequate base flow, thermal fluxes or flashy hydrology), which can have detrimental effects on ecosystems, even when water quality is not compromised (Coffman 2000). LID practices offer an additional benefit in that they can be integrated into the infrastructure and are more cost effective and aesthetically pleasing than traditional, structural storm water conveyance systems (EPA Website, http://www.epa.gov/owow/nps/lid/, 2009).

12.7 CHECKLIST AND DESIGN AIDS

All of the design criteria in this chapter should be followed. Key considerations include:

1. Provide stormwater quality treatment in accordance with the requirements in Section 12.2 and Table 12-3.
2. Provide treatment of the WQCV calculated in accordance with the USDCM, Volume 3.
3. Provide completed design forms from the USDCM, Volume 3, for selected structural BMPs.
4. Provide schematic details of selected structural BMPs.
5. Design the invert of the inflow pipe to the detention basin to be higher than the water quality level in detention basins.
6. Provide water quality outlet structure designs that minimize the number of perforation columns and keep the diameter of perforation below 2 inches.
7. Design forebays to effectively capture sediment and keep the outlet structure from clogging.
8. Select BMPs with ease of maintenance as a top priority.
9. Provide information on proper maintenance of the BMP(s), including frequency of required maintenance in the design report.
10. If proposed BMP(s) are not selected from the USDCM, Volume 3, then provide documented evidence that the BMP can satisfy the minimum technical requirements by meeting or exceeding similarly applicable BMP(s) in the USDCM, Volume 3.

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Permanent Seeding

Description: Permanent seeding is the planting of perennial vegetation such as trees, shrubs, vines, grasses, or legumes on exposed areas for final permanent stabilization. This is done to stabilize the soil by holding soil particles in place. It also serves to reduce stormwater runoff velocity, maintain sheet flow, protect the soil surface from erosion, promote infiltration of runoff into the soil, and improve the aesthetics of a disturbed area. This BMP is economical; adaptable to different site conditions; and allows selection of the most appropriate plant materials.

Pollutant Removal Capability/Effectiveness

Perennial vegetative cover from seeding has been shown to remove between 50 and 100 percent of total suspended solids from stormwater runoff, with an average removal of 90 percent.

Feasibility

Permanent seeding is appropriate where permanent, long-lived vegetative cover is the most practical, or most effective method of stabilizing the soil. It will work best in areas that are at final grade, or will not be re-graded for at least a year. It is also effective where soils are unstable because of their texture, structure, a high water table, high winds or a steep slope. Vegetation will grow most efficiently in areas where the topsoil was never stripped, or where it has been returned and incorporated into the soil surface.

Design Criteria

- In all cases soil piles shall be controlled by best management practices to control erosion and sediment from leaving the construction site whether by wind erosion or erosion by precipitation.
- Where a suitable planting medium is not present, topsoil should be imported and incorporated.
- Plan to plant when soil and weather conditions are most favorable for growth.
- Select appropriate grass seed and vegetation.
- After planting, apply straw mulch in the amount of 2 tons per acre.

Potential Benefits/Concerns

- There is a high erosion potential during establishment of vegetative cover, so application of mulch or soil retention blanket may be necessary.
- Areas that fail to establish growth will need to be reseeded.
- Seeding times are limited depending on the season.
- Vegetative growth needs stable soil temperature and soil moisture content during germination and early growth.
- Permanent seeding does not immediately stabilize soils so they may require other temporary erosion and sediment control methods.
- Establishing vegetation may require irrigation.
- Success depends on proper maintenance, climate and weather.

Distinguish Future Use

- Permanently seeded areas can be considered high- or low- maintenance areas.
- High-maintenance areas are mowed frequently, limed and fertilized regularly, and either receive intense use (e.g., athletic fields) or require maintenance to an aesthetic standard (e.g., home lawns).
- Select grasses for high-maintenance areas that are long-lived perennials that form a tight sod and are fine-leaved.
- Low-maintenance areas are mowed infrequently or not at all, and do not receive lime or fertilizer on a regular basis.

- Plants must be able to persist with minimal maintenance over long periods of time. Grass and legume mixtures are favored for these sites because legumes fix nitrogen from the atmosphere.
- Suitable sites include steep slopes, stream or channel banks, and “utility” turf areas such as road banks.

Grading and Shaping

- Grading and shaping may not be required where hydraulic seeding and fertilizing equipment is to be used.
- Overlot grading will often bring to the surface subsoils that have low nutrient value, little organic matter content, few soil microorganisms, rooting restrictions, and conditions less conducive to infiltration of precipitation.
- Vertical banks should be sloped to enable plant establishment.
- Grade and shape the slope so that equipment can be used safely and efficiently during seedbed preparation, seeding, mulching and maintenance of the vegetation.
- Concentrations of water that could cause excessive soil erosion should be diverted to a safe outlet.

Topsoil

- Topsoil should be friable and loamy, free of debris, objectionable weeds and stones, and contain no toxic substances that may be harmful to plant growth.
- Soil pH should be between 6.0 and 6.5 and can be increased with liming if soils are too acidic.
- Topsoil should be handled only when it is dry enough to work without damaging soil structure.
- A uniform application of 5 inches (unsettled) is recommended, but may be adjusted.

Cubic Yards of Topsoil Required to Attain Various Soil Depths		
Depth of Topsoil (Inches)	Volume Topsoil Per 1,000 per Square Feet	Volume Topsoil Per Acre
1	3.1	134
2	6.2	268
3	9.3	403
4	12.4	537
5	15.5	672
6	18.6	806

Seedbed Preparation

- The surface soil must be loose enough for water infiltration and root penetration.
- Tillage at a minimum should loosen the soil to a depth of 4 to 6 inches; alleviate compaction; incorporate topsoil, lime, and fertilizer; smooth and firm the soil; allow for the proper placement of seed, sprigs, or plants; and allow for the anchoring of straw or hay mulch if a crimper is to be used.
- Apply fertilizer in an amount to result in 40 pounds of available nitrogen per acre.

Costs

Seeding costs range from $200 to $1,000 per acre and average $400 per acre. Maintenance costs range from 15 to 25 percent of initial costs and average 20 percent.

Maintenance

- Inspection of the seeding application should be regularly performed.
- Reseed or replace areas that have eroded.
- If vegetation continually fails to grow, soil should be tested to determine if low pH or nutrient imbalances are responsible.

Sod Stabilization

Runoff Reduction BMP

Description: Sodding is a permanent vegetative cover used as an erosion control practice. It involves laying grass sod on exposed soils for stabilization. In addition to soil stabilization, sodding reduces the velocity of stormwater runoff, provides immediate cover for critical areas, and quickly stabilizes areas that cannot be seeded.

Pollutant Removal Capability/Effectiveness

Studies show that sod removes up to 99 percent of total suspended solids in runoff. This makes it a highly effective management practice for erosion and sediment control. The trapping efficiency, however, is highly variable depending on hydrologic, hydraulic, vegetation, and sediment characteristics.

Feasibility

Sodding can be used on any graded or cleared area that needs immediate cover to protect from erosion, and a permanent plant cover is needed immediately. Prime locations for sodding are residential or commercial lawns, buffer zones, stream banks and other waterways, swales, steeply sloped areas, filter strips, and areas where prompt use and aesthetics are important.

Design Criteria

Soil Preparation

- Fine-grade soil surface before laying down the sod.
- Clear surface of trash, woody debris, stones and clods larger than 1 inch.
- Soils should be tested to determine if amendments are needed, and amendments added if necessary.
- If topsoil is removed, it should be replaced. Do not lay sod on less than 4 inches of topsoil.

Installation

- Sod should be harvested, delivered, and installed within a period of 36 hours.
- Sod can be laid during times of the year when seeded grasses are likely to fail.
- Do not plant sod during very hot or wet weather.
- Sod should not be placed on slopes greater than 3:1 if they are to be mowed.
- Sod should be laid in strips perpendicular to the direction of water-flow as shown in Figure 12-1.
- Lay sod in a staggered, brick-like pattern, as shown in Figure 12-2.
- Don't overlap joints.
- Firmly staple the corners and middle of each strip, as shown in Figure 12-3. When ready to mow, drive stakes or staples flush with the ground.
- Jute or plastic netting may be pegged over the sod for further protection against washout during establishment.
- Roll, then irrigate sod and the top 4 inches of soil immediately after installation.
- Water sod frequently within the first few weeks of installation

Materials

- Sod used should be certified.
- Type of sod selected should be composed of plants adapted to site conditions.
- Sod composition should reflect both environmental conditions and future site uses.
- Sod should be machine cut at a uniform soil thickness of 15 to 25 mm at the time of establishment.
- Sod should be cut to the desired size.

Potential Benefits/Concerns

- Sod is more expensive than seed, and it is more difficult to obtain, transport and store.
- Soil preparation must be done carefully.
- Adequate moisture must be provided before, during and after installation for successful establishment.
- If sod is not adequately irrigated after installation, root dieback may occur because grass does not root rapidly and is subject to drying out.

Costs

Construction sodding averages between $0.20 per square foot and range from $0.10 to $1.10 per square foot; maintenance costs are approximately 5 percent of installation costs.

Maintenance

- Inspect sod frequently, especially after large storm events. The appearance of good sod is shown in Figure 12-4.
- Remove and replace dead sod.
- Mow new sod sparingly; mowing should not result in the removal of more than one third of the shoot.
- Grass height should not be cut to less than 2-3 inches.

Schematic Design of Sodding

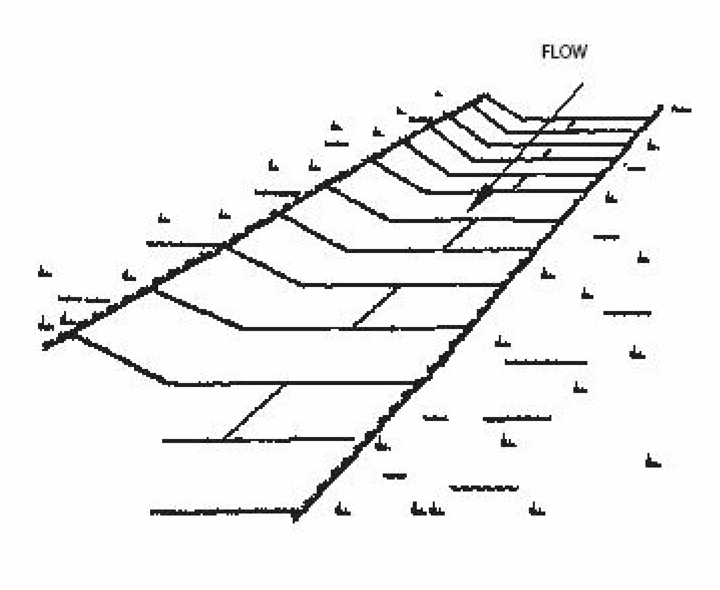

Figure 12-1 Strip Direction

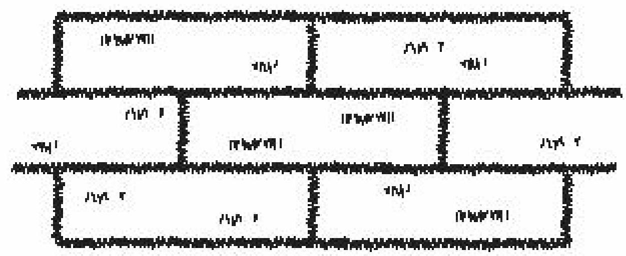

Figure 12-2 Sod Pattern

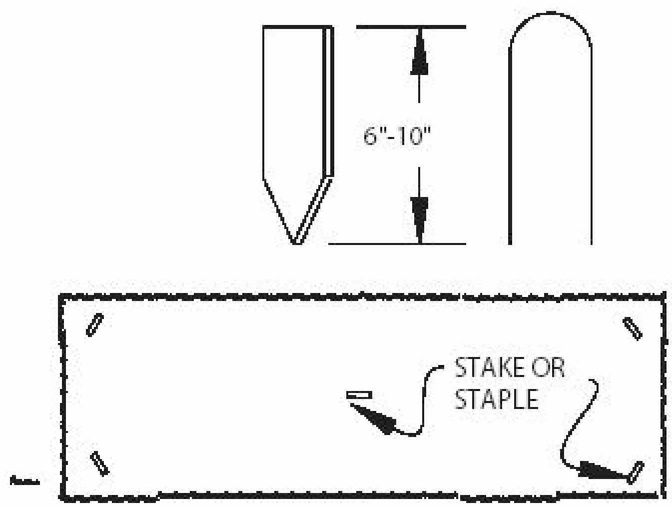

Figure 12-3 Staple of Stake Pattern

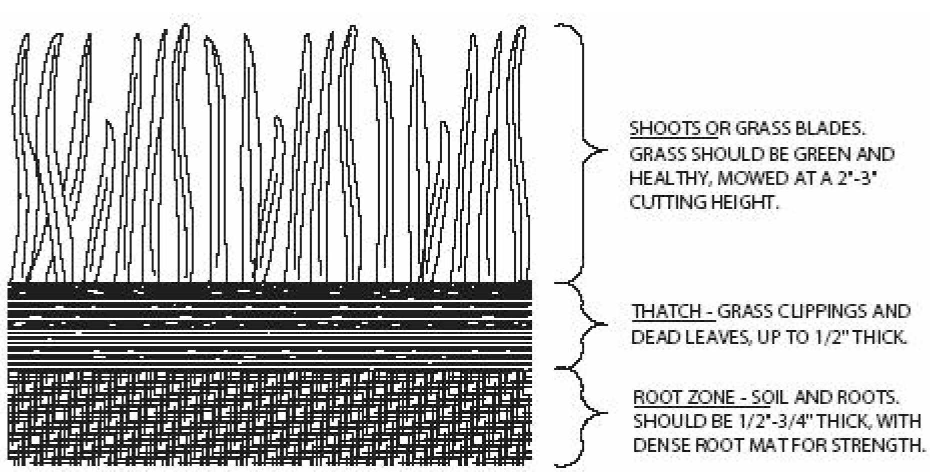

Figure 12-4 Appearance of Good Sod

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Grassed Swales

Runoff Reduction BMP

Description: Grassed swales are earthen conveyance systems, in which pollutants are removed from urban stormwater by filtration through grass and infiltration through soil. Swales should be designed with relatively wide bottoms to promote even flow through the vegetation and avoid channelization, erosion, or high velocities. Enhanced grassed swales, or biofilters, are vegetated open channels that utilize check dams and wide depressions to increase runoff storage and promote greater settling of pollutants.

Pollutant Removal Capability/Effectiveness

Grassed swales act to remove pollutants by the filtering action of grass, by settling, and in some instances, by infiltration into the subsoil. Conventional grassed swale designs have achieved mixed performance in removing particulate pollutants such as suspended solids and trace metals. They are generally unable to remove significant amounts of soluble nutrients. Biofilters that increase detention, infiltration and wetland uptake within the swale have the potential to substantially improve swale removal rates. Pollutant removal will be reduced if dry-weather (base) flow is present in the swale.

Feasibility

Swales can provide sufficient runoff control to replace curb and gutter in single-family residential subdivisions and on highway medians; however, their ability to control large storms is limited. Therefore, in most cases, swales must be used in combination with other BMPs downstream. Swale performance diminishes sharply in highly urbanized settings. Also, swales should generally not receive construction site runoff.

Design Criteria

Figure 12-5 shows a typical swale sections. The design criteria for swales include:

- Grassed swales can only be most effectively applied in areas where maximum flow rates are not expected to exceed 1.5 fps (Horner, 1988).
- The suitability of a swale at a site will depend on the area, slope and imperviousness of the contributing watershed as well as the dimensions and slope of the swale system.
- Swales generally do not have the capacity to control runoff effectively in areas where peak discharge exceeds 5 cfs or where velocity is over 3 fps.
- To decrease velocity, the swale should be designed to be as wide as available space allows.
- The high sediment loads from unstabilized construction sites can overwhelm the system.
- Grassed swales can be used in all regions of the country where climate and soils permit the establishment and maintenance of good vegetative cover.
- The performance of swales in removing pollutants may be reduced in regions with long, cold winters and snow melt conditions, particularly where salts and other de-icing chemicals are applied or where snow plowing scrapes the shoulder. In arid climates careful selection of plant species or occasional irrigation may be required.
- Swales may be less effective in regions with sandy soils (sandy soils make it difficult to maintain the side slopes of the swale.)
- To increase infiltration rates, longitudinal slopes should be as close to zero as possible and not greater than five percent (Schueler, 1987).
- A vertical stand of dense vegetation higher than the water surface is most effective (a minimum of six inches) (SEWRPC, 1991).
- Vegetation should be chosen based on the conditions expected in the swale (i.e., frequent inundation or prolonged periods of dry weather).
- In general, pollution removal capacity increases with contact time of runoff through the swale.

- Swale contact time varies with the depth, width and length of the swale as well as longitudinal slope and type of vegetation. Any one of these variables or sets of variables can be manipulated to meet water quality objectives.
- In addition, check dams can further increase contact time (Horner, 1988).

Use in Ultra-urban Areas: It is very difficult to prevent erosion in swales located in highly impervious, ultra-urban areas.

Retrofit Capability: Many residential developments and highways have existing grass channels. An attractive retrofit option is to install check dams to increase contact time and promote settling using portable weirs.

Stormwater Management Capability: Conventional grassed swales are primarily a stormwater conveyance system and rarely provide sufficient detention to attenuate storm flows. The exception is when detention storage is provided behind check dams in very long swale systems.

Potential Benefits/Concerns

Positive Impacts:

- When grassed swales are substituted for curbs and gutters, they can slightly reduce impervious areas, and more importantly, eliminate a very effective pollutant collection and delivery system
- Low slope swales can create wetland acreage.
- Unmowed swale systems that are not adjacent to roadways can provide valuable "wet meadow" habitat.
- Swales can act to partially infiltrate runoff from small storm events if underlying soils are not compacted.
- Swales eliminate curbs and gutters and provide some infiltration and habitat benefits.

Negative Impacts:

- Culverts may leach trace metals into runoff.
- Lawn fertilization may increase runoff nutrient levels.
- Possible impact on local groundwater quality.
- Standing water in residential swales will not be popular with adjacent residents for aesthetic reasons and because of potential safety, odor and mosquito problems.

Maintenance

- Mowing and periodic sediment clean out are the primary maintenance activities. In residential subdivisions, adjacent homeowners will undertake these responsibilities.
- Also, inspection after large storms for erosional failures and special maintenance should occur regularly.

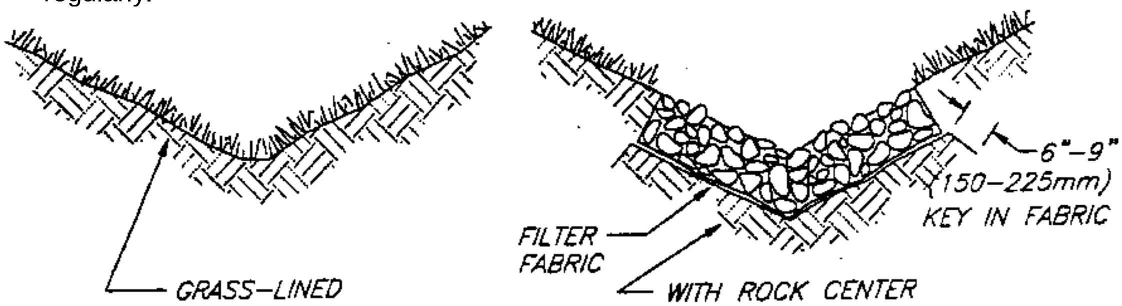

Figure 12-5 Schematic Design of Grassed Swales

Streambank Stabilization

Description: Stream bank stabilization BMPs are used to prevent stream bank erosion from high velocities and quantities of stormwater runoff. Stream channels, streambanks and associated riparian areas are dynamic and sensitive ecosystems that respond to changes in land use activity. Streambank and channel disturbance resulting from development activities can increase the stream's sediment load, which can cause channel erosion or sedimentation and have adverse effects on the system.

Pollutant Removal Capability/Effectiveness

The effectiveness of these methods is varied. Once established and properly maintained, erosion can be efficiently controlled.

Feasibility

It is appropriate to stabilize a streambed before any project that would disturb or occur within stream channels and their riparian areas. Any project that would disturb these riparian areas requires some type of stream bank stabilization.

Some typical methods include:

- Revegetation- The best way to prevent erosion and stabilize a stream bank is to keep the area vegetated.
- Armoring- Materials are applied to strengthen the stream bank. Natural materials such as logs are preferred since their shelter and decomposition help allow revegetation.
- Tree Revetments- A tree revetment, made by anchoring trees along a stream bank, is an inexpensive, effective way of stopping stream bank erosion.
- Riprap- Large angular stones placed along the stream bank or lake.
- Gabion- Rock-filled wire cages that are used to create a new stream bank.
- Reinforced Concrete- Concrete bulkheads and retaining walls that replace natural stream banks and create a non-erosive surface.
- Log Cribbing- Retaining walls built of logs to anchor the soils against erosive forces. Usually built on the outside of stream bends.

Design Criteria

Planning

- Stabilization should occur before any land development in the watershed area.
- Planning should take into account: scheduling; avoidance of in-stream construction; minimizing disturbance area and construction time period; using pre-disturbed areas; selecting crossing location; and selecting equipment.
- Avoid steep and unstable banks, highly erodible or saturated soils, or highly fractured rock.
- Preserving existing vegetation around a streambed provides water quality protection, soil stabilization, and riparian habitat.

Riprap

- Riprap is one of the most common streambed stabilization methods.
- It is often used on steep slopes built with fill materials that are subject to harsh weather or seepage.
- Place riprap over a filter blanket.
- Use either uniform size riprap or graded applied in an even layer throughout the stream.
- Figure 12-6 shows a typical riprap rock filter.

Potential Benefits/Concerns

- This BMP does not provide the same level of water quality or aesthetic benefits of vegetative practices.
- The method chosen should be designed by qualified professional engineers, which may increase project costs.
- The material costs may be expensive.
- Additional permits may be required for the structure.
- Wildlife habitats may be negatively impacted.
- Regularly inspect stream bank stabilization structures, especially after a large storm event.
- Repair damage as soon as possible to prevent further damage or stream bank erosion.
- Does not usually require as much maintenance as vegetative erosion controls.

Maintenance

Cost

The cost for these methods varies widely according to which method is chosen.

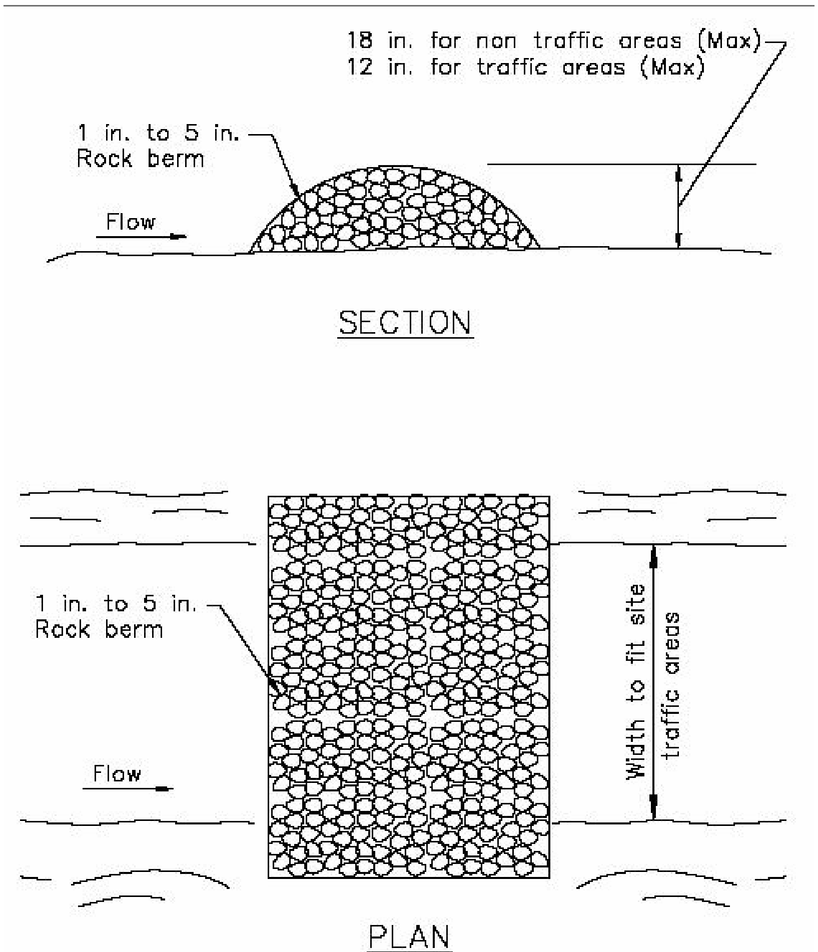

Figure 12-6 Rip Rap Rock Filter

Permeable Pavements

Runoff Reduction BMP

Description: Permeable pavement is an alternative to conventional pavement whereby runoff is diverted through a porous asphalt or concrete layer and into an underlying stone reservoir.

Pollutant Removal Capability/Effectiveness

Operating permeable pavement systems have been shown to have high removal rates for sediment, nutrients, organic matter, and trace metals. The majority of the removal occurs as the result of the exfiltration of runoff into the subsoil, and subsequent adsorption or straining of pollutants within the subsoil.

Pollutant removal mechanisms include adsorption, straining, and microbial decomposition in the subsoil below the aggregate chamber, and trapping of particulate matter within the aggregate chamber. In addition, up to 90% of the annual rain fall volume is diverted to groundwater rather than surface runoff (Schueler, 1987).

Feasibility

The use of permeable pavement is highly constrained, requiring deep and permeable soils, restricted traffic, and suitable adjacent land uses. Use of permeable pavement may be restricted in regions with colder climates, arid regions or regions with high wind erosion rates, and in areas of sole-source aquifers.

Design Criteria

Soils: Permeable pavement is not practical in soils with field verified infiltration of less than 0.5 inches per hour. Soil borings must be taken two to four feet below the aggregate to identify any restricting layers.

Area: Most permeable pavement sites are less than ten acres in size. This primarily reflects the perceived economic and liability problems associated with larger applications.

Slope: Less than five percent.

Depth to Bedrock and Water Table: Three feet minimum clearance from bottom of system.

Sole-Source Aquifer: Use of permeable pavement should be evaluated carefully in regions where water is supplied by a single aquifer.

Traffic Volumes: Permeable pavement is not recommended for most roadways and cannot withstand the passage of heavy trucks. Typically, permeable pavement is recommended for lightly used satellite parking areas and access roads.

Sediment Inputs: Permeable pavement is not advisable in areas expected to provide high levels of off-site sediment input (e.g., upland construction, sparsely vegetated upland areas and areas with high wind erosion rates)

Cold Climates: While the standard permeable pavement design is believed to withstand freeze/thaw conditions normally encountered in most regions of the country, the permeable pavement system is very sensitive to clogging during snow removal operation (e.g., application of sand and de-icing chemicals and scraping by snow plows).

Potential Benefits/Concerns

Positive Impacts:

- Permeable pavement can divert large volumes of potential surface runoff to groundwater recharge and, in some cases, provide even greater recharge than predevelopment conditions (OWML, 1986).
- Permeable pavement can reduce downstream bank-full flooding.
- Provides stormwater quantity and quality treatment on-site, thereby protecting woodland, wetland, and stream valleys elsewhere on the site (Cahill, et. al., 1991).

Negative Impacts:

- Slight to moderate risk of groundwater contamination depending on soil conditions and aquifer susceptibility.
- Possible transport of hydrocarbons from vehicles and leaching of toxic chemicals from asphalt/or binder surface.
- The high failure rate of permeable pavement sharply limits the ability to meet watershed stormwater quality and quantity goals.

Maintenance

Quarterly vacuum sweeping and/or jet hosing is needed to maintain permeability. Field data however indicate that this routine maintenance practice is not frequently followed.

Permeable pavement sites have a high failure rate (75 %). Failure is due to partial or total clogging of the area that occurs immediately after construction, when permeable asphalt is clogged by sediment and oil, or when pavement is resurfaced with non-permeable materials.

Stormwater Planter

Runoff Reduction BMP

Description: Planter boxes are elevated structures containing plants or trees that may be used as stormwater control devices in urban environments. As part of a disconnection strategy, roof downspouts may be directed to vegetated planter boxes to store and filter stormwater. Trees in planter boxes intercept rainfall before it can be converted to stormwater. Planter boxes offer “green space” in tightly confined urban areas that provide a soil/plant mixture suitable for stormwater capture and treatment.

Pollutant Removal Capability/Effectiveness

Water quality benefits are similar to those for bioretention cells. 50% phosphorus removal is achieved for the first 0.5” of runoff from impervious areas that enters the planter box. Planter boxes contribute to pollutant load reductions by minimizing the volume of stormwater generated. Rainfall is retained and stored in the soil within the planter boxes and the planted vegetation intercepts rainfall and evapotranspirates moisture. Pollutant concentration reductions will also occur as the stormwater infiltrates through the planter box soil. Pollutants adsorb and are absorbed by the soil particles. Aerobic decomposition and chemical precipitation will also decrease pollutant concentrations within the soil matrix. A determination of pollutant load reductions will require an analysis of adsorption and absorption rates for soluble pollutants as well as decomposition and precipitation rates. Reductions in particulates and suspended solids will be achieved by physical removal when filtering through the aggregate.

Feasibility

Planter boxes are most commonly used in urban areas adjacent to buildings and along sidewalks. Locations close to roof downspouts are preferable when used a part of a disconnection program.

Design Criteria

Planter boxes may be constructed of any durable material. When built adjacent to buildings as a receptacle for downspout runoff, they are often constructed of the same material as the building. Otherwise they may be constructed of concrete to blend in with the sidewalk or metal when they are stand-alone units. An appropriate soil mix is also necessary to ensure plant growth and vitality. Indigenous plants and vegetation are preferable to ease maintenance. Underdrains can be installed to connect planter boxes to an adequate conveyance system. Observation/cleanout wells should be installed if underdrains are used. Planter boxes typically consist of:

- Planter Box Construction (concrete)
- Vegetation Planting
- Soil Media
- 4” diameter perforate underdrain pipe

Potential Benefits/Concerns

Positive Impacts:

- Improve water quality characteristics
- May reduce peak discharge and runoff volume
- Increase aesthetics
- Easily retrofitted

Maintenance

Maintenance activities entail routine inspections of the planter box structure and the underdrain. Soils also need to be inspected to evaluate root growth and channel formation within the soil matrix.

To ensure proper performance, visually inspect that stormwater is infiltrating properly into the planter box soil and that there is discharge from the underdrain during large wet weather events. Water ponding in a planter box for more than 24 hours may indicate operational problems. If excessive water ponding is observed, corrective measures include inspection for soil compaction and for underdrain clogging. The soil media may need to be turned or tilled to improve infiltration and the vegetation replaced. Backflushing the underdrain may be able to remove obstructions. If these efforts are unsuccessful, the soil media and underdrain may need to be removed and replaced.

Dry Well

Description: A dry well typically consists of a pit filled with large aggregate such as gravel or stone. Alternately, it may consist of a perforated drum placed in a pit and surrounded with stone. Dry wells capture and infiltrate water from roof downspouts or paved areas. The surface is typically at or just below existing grade. It may be covered by grass or other surface.

Pollutant Removal Capability/Effectiveness

Water quality benefits are similar to those for infiltration trenches. Phosphorus removal efficiencies for infiltration trenches are:

- 50% removal for trenches that capture 0.5" of runoff from the impervious area.
- 65% removal for trenches that capture 1.0" of runoff from the impervious area.

Feasibility

Dry wells are suitable for treating small impervious areas (as an alternative to infiltration trenches) and may be useful on steeper slopes where trenches or other facilities cannot be installed. Dry wells are particularly suited to treat runoff from residential driveways or rooftop downspouts. It is important to avoid installation in large areas with high sediment loads and in soils with limited permeability. Dry wells are not appropriate for treating runoff from large impervious surfaces such as parking lots.

Design Criteria

Design guidelines for dry wells are similar to those for infiltration trenches.

Potential Benefits/Concerns

Positive Impacts:

- Can be used on steeper slopes
- May obtain points towards LEED credits

Negative Impacts:

- May experience nutrient leaching in beginning stages of applying soil amendments.

Maintenance

Dry wells are typically employed in single-family homes; maintenance is usually the responsibility of the homeowner. Maintenance consists of debris removal from the rain gutters and dry well surface (or chamber, depending on design), and minor parts replacement.

Performance and Inspection: Inspect the dry well and rain gutters for debris accumulation. Perform this inspection bi-annually in spring and fall and after large storm events.

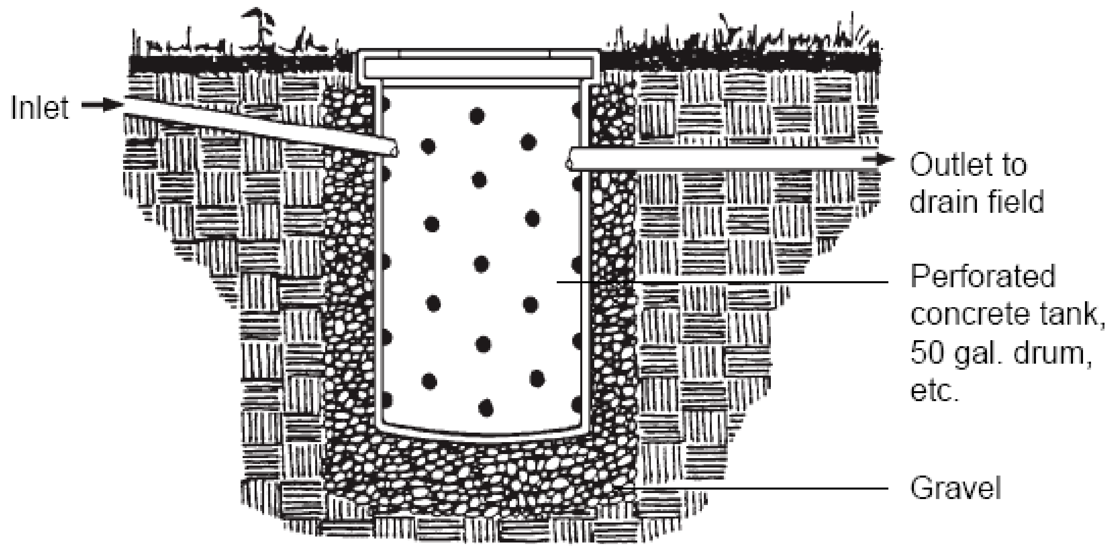

Figure 12-7 Dry Well

Tree Box Filter

Runoff Reduction BMP

Description: Tree box filters are mini bioretention areas installed beneath trees that can be very effective at controlling runoff, especially when distributed throughout the site. Runoff is directed to the tree box, where it is cleaned by vegetation and soil before entering a catch basin. The runoff collected in the tree-boxes helps irrigate the trees. Tree boxes are typically proprietary structures.

Pollutant Removal Capability/Effectiveness

Tree boxes have proven to be an effective stormwater management too. Studies show tree boxes removing greater that 90% TSS, 70% hydrocarbons (TPH-D), and greater than 25% of dissolved inorganic nitrogen (DIN) (UNH, 2006). Another study reveals that tree boxes remove up to 95% of TSS, 92% of total phosphorus, 76% total nitrogen and 91% of heavy metals (Coffman Siviter, 2007).

Feasibility

Tree boxes are suitable for small drainage areas. They can fit in most landscape schemes.

Design Criteria

Design criteria are usually determined by the manufacturer. Therefore, when designing a tree box filter, determine the manufacturer and utilize the design guide specified.

Potential Benefits/Concerns

Positive Impacts:

- Enhance water quality and water quantity characteristics.
- Increase aesthetics.
- Reduce heat island effect.

Maintenance

In general, maintenance activities include annual routine inspection and maintenance. Also, the first two years of maintenance are typically included with the purchase of single and multiple-unit tree box filters. These would include removal of trash, debris and sediment, replenishment of the mulch, and care or replacement of plants. During extreme droughts the plants may need to be watered in the same manner as any other landscape material. In the event of a chemical spill all of the soil and plants should be removed disposed of properly and replace with new uncontaminated filter media and plants. Tree box filters require low maintenance and a small drainage area with respect to surface area.

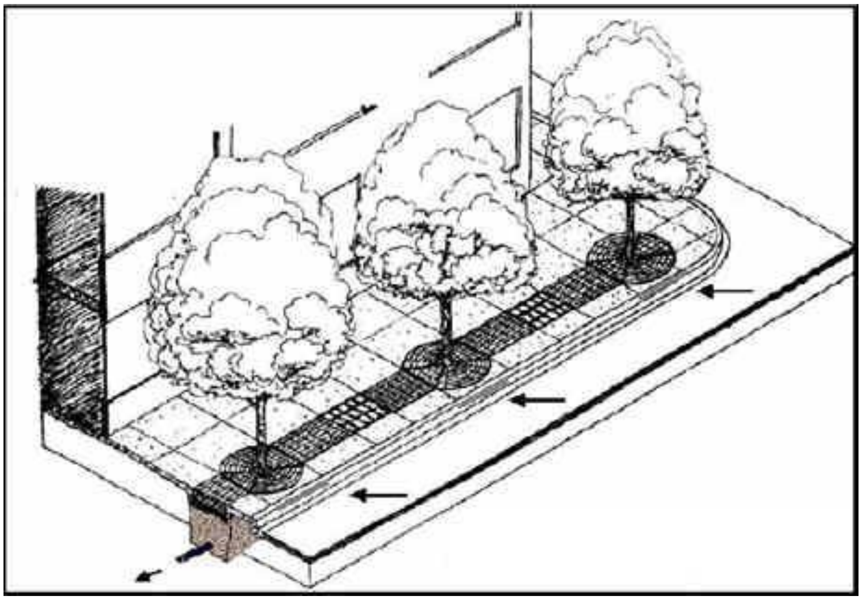

Figure 12-8 Tree Box

Green Roof

Description: Green roofs are contained vegetation systems living on top of buildings or structures. This green space involves systems where plants are not planted in the ground but in space below, at, or above grade (Green Roofs for Healthy Cities). There are two main types of green roofs: extensive and intensive. Extensive green roofs are shallow, usually with 4 inches of substrate, and do not typically support a large diversity of plant species because of root zone limitations. Intensive green roofs are more like rooftop gardens with deep substrate (from 4 inches to several feet) and a wide variety of plants.

Pollutant Removal Capability/Effectiveness

Water quality data for green roofs are not yet robust enough to provide meaningful conclusions about pollutant removal. By reducing volume, green roofs have the de facto capability to reduce pollutant loads; however, on a concentration basis more data are needed to better define effectiveness.

Feasibility

Most existing buildings are not designed to withstand the additional weight loading for intensive roofs. For this reason, they are typically limited to new construction. Extensive green roofs are shallower and generally much better suited to the structural capabilities of existing buildings and therefore, are installed more often. Because of this, extensive green roofs are the focus of this design guidance. The design of a green roof involves many disciplines in addition to stormwater engineers, including structural engineers, architects, landscape architects, horticulturists, and others. This Fact Sheet is intended only to provide an overview of green roof information relative to stormwater quality and quantity management that is applicable in the Rocky Mountain area.

As Low Impact Development (LID) strategies have been emphasized increasingly throughout the U.S., green roofs have been implemented in some parts of the country, most frequently in areas with humid climates and relatively high annual rainfall. Although there are some green roofs in Western States, they have not been widely installed. There is research in progress regarding the best design approach and plant list for the Colorado front range climate.

Wyoming's low annual precipitation, low average relative humidity, high solar radiation due to elevation, high wind velocities and predominantly sunny days make growing plants on a roof more difficult than in other climates. Because of this, plant selection, growing medium, and supplemental irrigation requirements are key considerations for which design criteria continue to evolve.

Design Criteria

Design Guidelines and Maintenance Manual for Green Roofs in the Semi-Arid and Arid West, prepared by the University of Colorado Denver with input from UDFCD, should be used as a more comprehensive design and maintenance document. This document is available at www.growwest.org.

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Wet Pond

Description: Conventional wet ponds have a permanent water pool to treat incoming stormwater runoff. In enhanced wet pond designs, a forebay is installed to trap incoming sediments where they can be easily removed; a fringe wetland is also established around the perimeter of the pond.

Pollutant Removal Capability/Effectiveness

Conventional wet ponds provide moderate to high removal of both particulate and soluble urban stormwater pollutants. Reliable removal rates can be achieved with pool sizes ranging from 0.5 to 1.0 inches of runoff per impervious acre.

Pollutant Removal Mechanisms: Achieved by gravitational settling, algal settling, wetland plant uptake and bacterial decomposition (Driscoll, 1983). The degree of pollutant removal is a function of pool size in relation to contributing watershed area.

Review of Monitoring Studies: The pollutant removal capability of conventional wet ponds is well documented with over twenty performance monitoring studies in publication. Reported sediment removal typically ranges from 50-90%. Total phosphorus removal ranges from 30-90%. Removal of soluble nutrients ranges from 40-80%. Moderate to high removals of trace metals, coliforms and organic matter are frequently reported.

Factors Influencing Pollutant Removal:

Positive Factors

- Pretreatment by sediment forebay (Livingston, 1989).
- Permanent pool, 0.5 - 1.0 inches per impervious acre treated (6,15).
- Fringe wetlands.
- Shallow wetlands and/or extended detention may improve removal efficiencies (Adams, et. al., 1983).
- High length to width ratios.

Negative Factors

- Small pool size (Driscoll, 1983).
- Fecal contribution from large waterfowl populations (Wu, et. al.,1988).
- Short-circuiting and turbulence (Martin, 1988).
- Sediment phosphorus release.
- Extremely deep pool depths (greater than 10 feet).
- Snowmelt conditions and/or ice (Oberts, et. al., 1989).

Feasibility

Wet ponds can be utilized in both low and high visibility development situations if contributing watershed area is greater than ten acres and/or a reliable source of baseflow exists. Wet pond designs are not generally useful in arid regions where evapotranspiration significantly exceeds precipitation on an annual basis. Also, the size of the pool will need to reflect the prevailing climate and runoff frequency for a particular region. Ponds can be used in colder northern climates, but their performance declines slightly during ice and snowmelt runoff conditions. This practice may not be effective in more arid areas. Applications in Wyoming will need to be carefully chosen.

Design Criteria

Contributing Watershed Area: Contributing watershed areas greater than ten acres and less than one square mile are generally suitable for wet ponds.

Baseflow: Dry-weather baseflow is needed to maintain pool elevations and prevent pool stagnation.

Available Space: Wet ponds and associated buffer/setbacks can consume from one to three percent of total site area.

Development Situations: Very useful in both low and high visibility commercial and residential development applications.

Use in Ultra-urban Areas: Use in ultra-urban areas is fairly limited due to space constraints, but can provide an attractive urban amenity if open space or parkland is available.

Retrofit Capability: Occasionally used for stormwater retrofits, particularly within dry stormwater basins (Schueler, et. al., 1991). Often used in combination with wetlands or extended detention treatment techniques.

Stormwater Management Capability: Most wet ponds can provide two-year stormwater quantity control, in addition to quality control.

Potential Benefits/Concerns

Positive Impacts:

- Creation of wetland features.
- Creation of aquatic and terrestrial habitat (particularly for waterfowl).
- Creation of a warm-water fishery.
- High community acceptance and landscaping values (Adams, et. al., 1983).
- Pollutant removal and downstream channel protection.

Negative Impacts:

- Downstream warming (Galli, 1991). May not be appropriate on streams with cold water fisheries.
- Upstream channels may be impacted when wet ponds serve large drainage areas (> 250 acres) (Schueler, 1991).
- Potential loss of wetlands, forest and floodplain habitat associated with poor site selection for the pool (Schueler, 1991).
- Downstream shifts in trophic status (Galli, 1988).
- Limited risk of ground water quality impacts over the long term; all studies to date indicate that wet ponds do not significantly contribute to ground water contamination (USEPA, 1991).
- Potential hazard for nearby residents due to the presence of standing water. The inclusion of a shallow safety bench around the pond may reduce potential hazards. Additionally, growth of dense vegetation (cattails, willows, etc.) will limit access and hazards to residents.
- Provide areas for insect nuisances such as mosquitos that will need control.

Maintenance

Wet ponds have a modest maintenance burden, consisting primarily of inspections, mowing of the embankment and buffers, and removal of sediment, trash and debris from the forebay. All studies to date indicate that pond sediments meet sludge toxicity limits and can be safely land filled.

Factors Influencing Longevity: Well-designed wet ponds can function for twenty years or more and very few conventional ponds have ever failed to provide some water quality benefit. Performance will decline over time, however, unless regular sediment clean out is undertaken. Factors influencing the longevity of wet ponds include:

- Installation of a sediment forebay (Schueler, 1992).
- Regular (2 - 5 year) sediment clean-outs (Schueler and Helfrich, 1988).
- Reverse-slope pipes.
- On-site sediment disposal area.
- Use of concrete riser/barrels rather than corrugated metal pipe.

Schematic Design of Wet Pond System

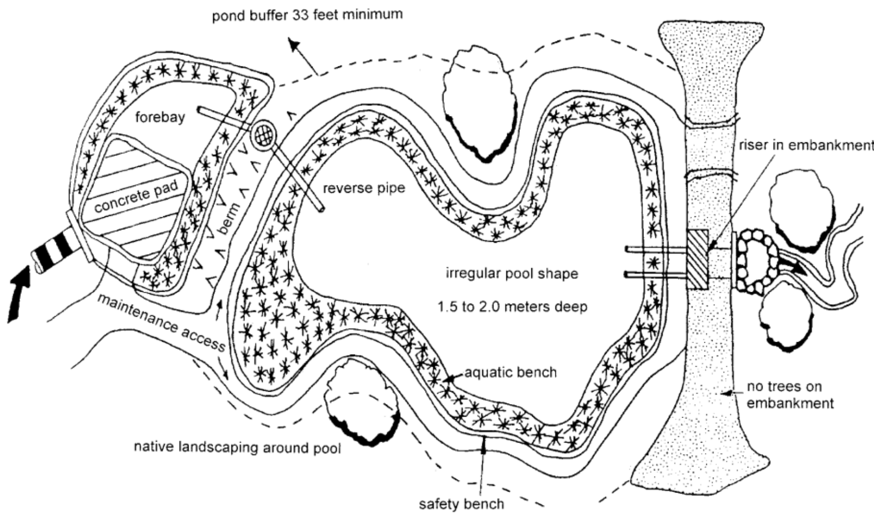

Figure 12-9 Wet Pond

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Sand Filter

Description: Sand filters are a relatively new technique for treating stormwater, whereby the first flush of runoff is diverted into a self-contained bed of sand. The runoff is then strained through the sand, collected in underdrain pipes and returned back to the stream or channel. Storage is generally calculated on the runoff volume of 0.5 inches of rainfall per impervious acre (Debo and Reese, 1995). Sand filters may be an "unconfined," sand-filled trench with a perforated underdrain. There are also "confined" systems where the filter medium is contained in a concrete vault with a drain at the bottom of the vault.

Pollutant Removal Capability/Effectiveness

Sand filter removal rates are high for sediment and trace metals, and moderate for nutrients, BOD and fecal coliform. The untested peat sand filter is projected to achieve significantly higher removal rates.

Pollutant removal is primarily achieved by straining pollutants through the filtering medium (i.e., sand or peat) and by settling on top of the sand-bed and/or pretreatment pool. Additional nutrient removal can be accomplished by plant uptake if the filter has a grass cover crop. Enhanced sand filters utilize layers of peat, limestone, leaf compost and/or topsoil, and may also have a grass cover crop. The adsorptive media of enhanced sand filters is expected to improve removal rates. In addition, sand-trench systems have been developed to treat parking lot runoff.

Feasibility

Because sand filters are a self-contained man-made soil system, they can be applied to most development sites and have few constraining factors. Sand filters and peat sand filters can be used to treat stormwater runoff from small in-fill developments and from small parking lots (i.e., gas stations, convenience stores). Sand filters have been successfully applied in Texas, Florida, Maryland, Delaware and Washington, DC. Performance of sand filters in colder climates is not well documented. It is expected that sand filters would lose some or all of their filtering ability if they freeze. Once thawed, they should function normally. Sand filters have a limited ability to reduce peak discharges; they are usually designed solely to improve water quality. Sand filters may be easily adapted into flood control BMPs.

Design Criteria

Climate: Sand filters are a very adaptable practice; they can be used on areas with thin soils, high evaporation rates, low soil infiltration rates, and limited space (City of Austin, 1988).

Watershed Size: The upper limit on sand filters appears to be about fifty acres; however, most have a contributing watershed between a half and ten acres.

Head Requirements: Two to four feet of available head needed for most off-line sand filter applications.

Potential Benefits/Concerns

Positive Impacts:

- Sand filters are useful in watersheds where concerns over groundwater quality prevent use of infiltration.
- Disposal of surface sediments from sand filters does not appear to be a problem.
- Testing by the Austin Department of Public Works indicates that the sediments are not toxic and can be land filled.
- Underground filters fit well into urban areas with restricted space (NCSU, 1998).
- Sand filters have very few environmental concerns because they are an off-line self-contained system.

Negative Impacts:

- Larger sand filter designs, without grass cover, may not be attractive in residential areas. The surface of sand filters can be extremely unattractive; some sand filters have caused odor problems.
- The concrete walls that surround the sand filter represent a safety hazard and thus should be fenced.
- Sand filters generally function only as a stormwater quality practice and do not provide detention for downstream areas.

Maintenance

Sand filters require frequent manual maintenance, primarily raking, surface sediment removal, and removal of trash, debris and leaf litter. Sand filters appear to have excellent longevity due to their off-line design and the high porosity of sand as a filtering media; however, relatively simple but frequent maintenance is required to maintain performance.

Most of the maintenance for sand filters is done by manual rather than mechanical means; consequently, the design should be oriented to make access and manual sediment removal as efficient as possible (City of Austin, 1991).

Schematic Design of Sand Filter System

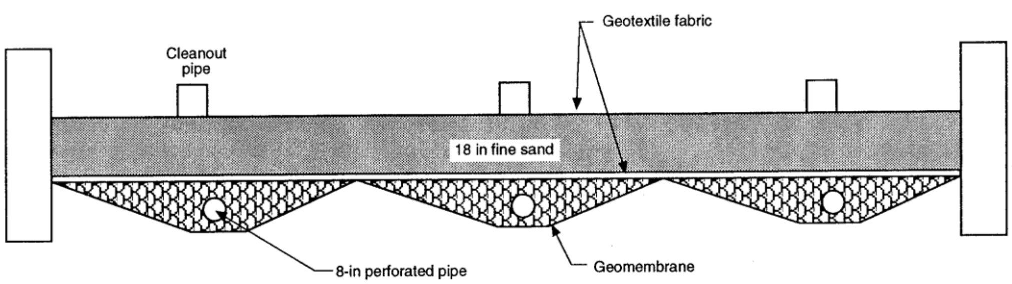

Figure 12-10 Sand Filter System

Bioretention Cells

Description: Bioretention cells are shallow depressed areas filled with an engineered soil mix and planted with trees, shrubs and other herbaceous vegetation. They are designed to capture and temporarily store stormwater runoff in the engineered soil mix, where it is subjected to the hydrologic processes of evaporation and transpiration, before being conveyed back into the storm drain system through an underdrain or allowed to infiltrate into the surrounding soils.

Pollutant Removal Capability/Effectiveness

Bioretention uses multiple treatment processes to remove pollutants, including sedimentation, filtering, absorption, evapotranspiration, and biological uptake of constituents.

Feasibility

Bioretention areas can be used to “receive” stormwater runoff on a wide variety of development sites, including residential, commercial and institutional development sites in rural, suburban and urban areas. They are well suited to “receive” stormwater runoff from nearly all small impervious and pervious drainage areas, including local streets and roadways, highways, driveways, small parking areas and disturbed pervious areas (e.g., lawns, parks, community open spaces).

Design Criteria

- Bioretention cells should be located within areas with hydrologic soil groups A or B. If located within hydrologic soil groups C or D an underdrain should be considered or an alternative stormwater technique such as wetlands or swales.
- Flat terrains may cause create ponding within bioretention cell. Ensure native underlying soils will enable the cell to drain completely within 48 hours.
- Ensure that there is 2’ clearance between the distance of the bottom of the bioretention cell and the water table.
- A head (ponding depth) between 6-12” is recommended.
- Depth of planter should be at least 36”.
- The insitu soils should have an infiltration rate of at least 0.5” inches per hour.
- A slope to 6% may be used, however a flatter slope is recommended.

Potential Benefits/Concerns

Positive Factors

- Create habitat and wild life diversity.
- Increased aesthetics.
- Achieve water quantity control.
- Achieve water quality benefits.
- Groundwater recharge.
- May be easily retrofitted to existing designs.

Negative Factors

- Filter material can become clogged. The backfill soil may need to be replaced.

Costs

When compared with other low impact development practices, bioretention areas have a moderate construction cost, a moderate maintenance burden and require a relatively small amount of surface area.

Maintenance

- Moderate maintenance is required.
- Collected trash and debris that are trapped within the cell.
- Trimming and mowing.
- May need replacement of mulch and soil.

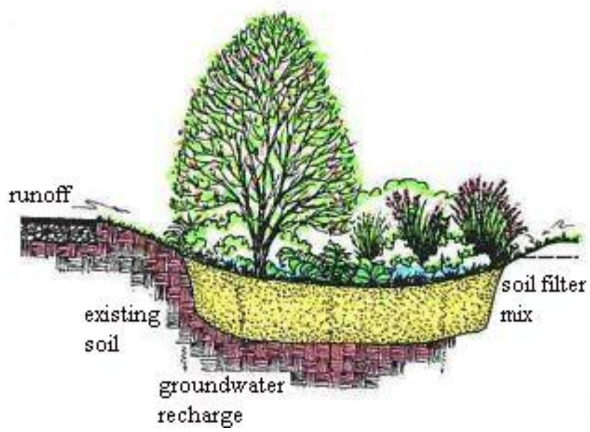

Figure 12-11 Bioretention Cell

Infiltration Trenches

Description: A conventional infiltration trench is a shallow, excavated trench that has been backfilled with stone to create an underground reservoir. Stormwater runoff diverted into the trench gradually exfiltrates from the bottom of the trench into the subsoil and eventually into the water table. Enhanced infiltration trenches have extensive pretreatment systems to remove sediment and oil. Both types of trenches require on-site geotechnical investigations to determine appropriate design and location.

Pollutant Removal Capability/Effectiveness

Although actual performance data on conventional infiltration trenches is scarce, trenches are believed to have high capability to remove particulate pollutants and a moderate capability to remove soluble pollutants. Pollutant removal mechanisms include adsorption, straining and microbial decomposition in the soil below the trench and trapping of particulate matter within pretreatment areas (i.e., grass filter strips, sump pits and plunge pools) (Schueler, 1987).

Feasibility

The application of trenches, like other infiltration practices, is severely restricted by soils, water table, slope and contributing area conditions. These conditions must be carefully investigated in the field before proceeding with design. The widespread use of infiltration trenches may be limited in areas where the ground commonly freezes or more arid climates where wind erosion may introduce a significant sediment load. Infiltration trenches are also less effective in regions where soils are predominantly clays or silts.

Design Criteria

Figure 12-12 shows a typical Infiltration Trench.

Volume: Trenches may be designed to accept the "first flush" volume (½ runoff per acre of impervious surface) or for larger volumes of runoff. A design variation is a dry well to control small volumes of runoff, such as roof top runoff.

Ground Water: Trenches may not be easily adaptable in regions where ground water is used locally for human consumption or in areas where particularly hazardous pollutants may be present.

Sediment: Conventional trenches may not be advisable on sites expected to provide high levels of sediment.

Climate: Trenches may not perform well in regions with long, cold winters and deep freeze-thaw levels. Trenches may not be appropriate in arid regions with sparse vegetative cover in upland areas that might contribute high sediment levels.

Stormwater Management Capability: Some trench designs can provide stormwater quantity requirements; however, most trenches function only as water quality BMPS.

Soils: Trenches are not practical in soils with field-verified infiltration rates of less than 0.5 inches per hour (particularly silty or clayey soils). Soil borings should be taken well below the proposed bottom of the trench to identify any restricting layers (MDE, 1983).

Area: Maximum contributing drainage area to an individual trench should not exceed five acres.

Slope: The effectiveness of surface trenches is sharply reduced if slopes are greater than five percent.

Depth to Bedrock and Depth to Water Table: Three feet of clearance from bottom of trench to the water table is recommended.

Potential Benefits/Concerns

Positive Impacts:

- Groundwater recharge.
- Reduction in downstream bank full flooding events.
- Some reduction of peak stormwater discharge.

Negative Impacts:

- Slight to moderate risk of groundwater contamination depending on soil conditions. Infiltration trenches or basins are not recommended in areas where the potential of significantly polluted surface runoff exists.
- No habitat is created.
- High failure rates of conventional trenches sharply limit the ability to meet stormwater and water quality goals at the watershed scale.
- May cause an increase in the groundwater table, resulting in flooding of structures with basements. This is more likely to be a problem in areas with existing high water table.
- Thus far, conventional trenches have proved to have short life spans. Slightly over half partially or totally fail within five years of construction. Longevity could be greatly improved through the utilization of enhanced trenches (i.e., runoff pretreatment, better geotechnical evaluation and regular maintenance). An underdrain installed during construction of an infiltration trench may increase longevity by allowing conversion to a sand filter should the trench fail due to poor exfiltration. The drain would remain capped until the trench failed.

Maintenance

To enhance longevity and maintain performance, trenches and associated pretreatment systems do require significant maintenance. Most conventional trenches do not appear to be regularly maintained in the field and thus many will require costly rehabilitation or replacement to maintain their function.

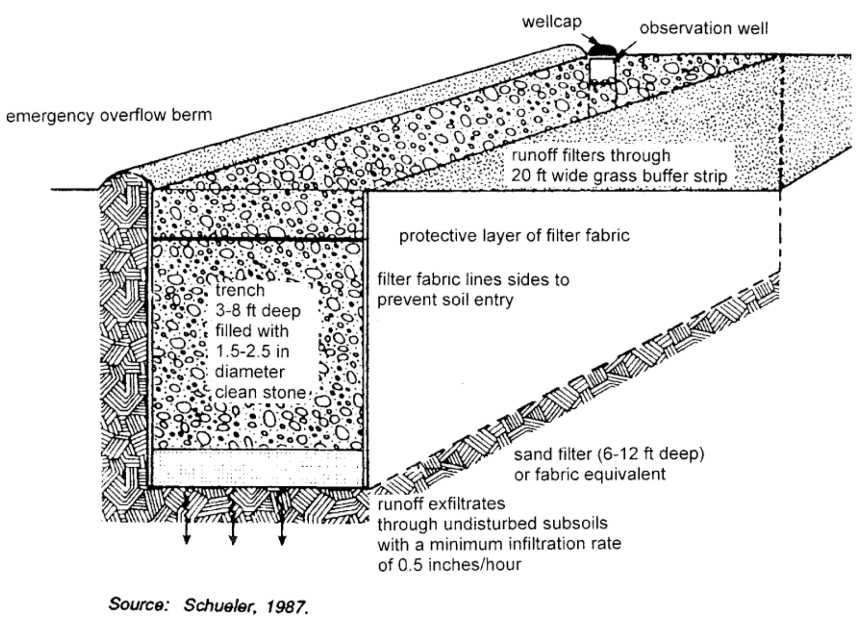

Figure 12-12 Infiltration Trench

Infiltration Basins

Description: Infiltration basins are impoundments where incoming stormwater runoff is stored until it gradually exfiltrates through the soil of the basin floor.

Pollutant Removal Capability/Effectiveness

No performance data on infiltration basins is available; however, they are presumed to have the same general removal efficiencies reported for infiltration trenches: high removal for particulate pollutants and moderate removal for soluble pollutants. As with other infiltration systems, removal is accomplished by adsorption, straining, and microbial decomposition in the basin subsoils as well as the trapping of particulate matter within pretreatment areas (Schueler, 1987). Drainage of the basin should occur within a minimum of 72 hours to maintain aerobic conditions and promote microbial removal of pollutants.

Feasibility

The application of basins is restricted by numerous site factors (soils, slope, water table and contributing watershed area). Infiltration basins may not be applicable in areas of cold winters, arid growing seasons or impermeable soils.

Design Criteria

A typical Infiltration Basin is shown in Figure 12-13.

Soils: Basins are not feasible at sites with field-verified soil infiltration rates of less than 0.5 inches/hour. Soil borings should be taken well below the proposed bottom of the basin to identify any restricting layers (MDE, 1983).

Contributing Watershed Area: Normal contributing drainage area ranges from two to fifteen acres. Larger drainage areas are not generally recommended.

Depth to Bedrock/Seasonally High Water Table: Minimum of three feet.

Sole-Source Aquifers: Regions with sole-source aquifers may not be suitable.

Pretreatment: Basins are not recommended unless upland sediment inputs can be pretreated.

Land Use: Some caution should be exercised when applying a basin in a watershed with a risk of chronic oil spills or other hazardous materials.

Stormwater Management Capability: In some instances, a basin can provide stormwater management detention, but it is not generally recommended.

Potential Benefits/Concerns

Positive Impacts:

- Groundwater recharge helps to maintain dry-weather flows in streams.
- Reduction in downstream bank full flooding events. Partial replication of predevelopment hydrology. (Note: The short lifetimes of basins as currently designed suggest that the positive hydrological and water quality impacts may not be realized in practice.)

Negative Impacts:

- Slight to moderate risk of local groundwater contamination (particularly if contributing watershed is industrial or has heavy vehicular petroleum wash off).

- Infiltration basins provide some habitat value, but this is quite modest in comparison to that provided by pond systems. Failed basins provide better habitat than functioning basins.
- Infiltration basins in close proximity to streams or lakes may degrade water quality if there is a rapid hydraulic connection between the basin and nearby surface water.
- May cause an increase in the local ground water table resulting in flooding of nearby basements. This is most likely to be a problem in areas of high existing ground water table.
- Infiltration basins do not have long life spans. Sixty to one hundred percent of basins studied could no longer exfiltrate runoff after five years. Major design refinement and site investigation will be required to achieve sufficient longevity. Installation of a back-up underdrain may extend the life of a basin by essentially converting it into a sand filter. The drain, normally capped, may be opened when exfiltration is no longer effective.

Maintenance

Regular maintenance activities apparently cannot prevent rapid clogging of infiltration basins. Once clogged, it has been very difficult to restore their original function; thus, many have been converted to retention basins or wetlands.

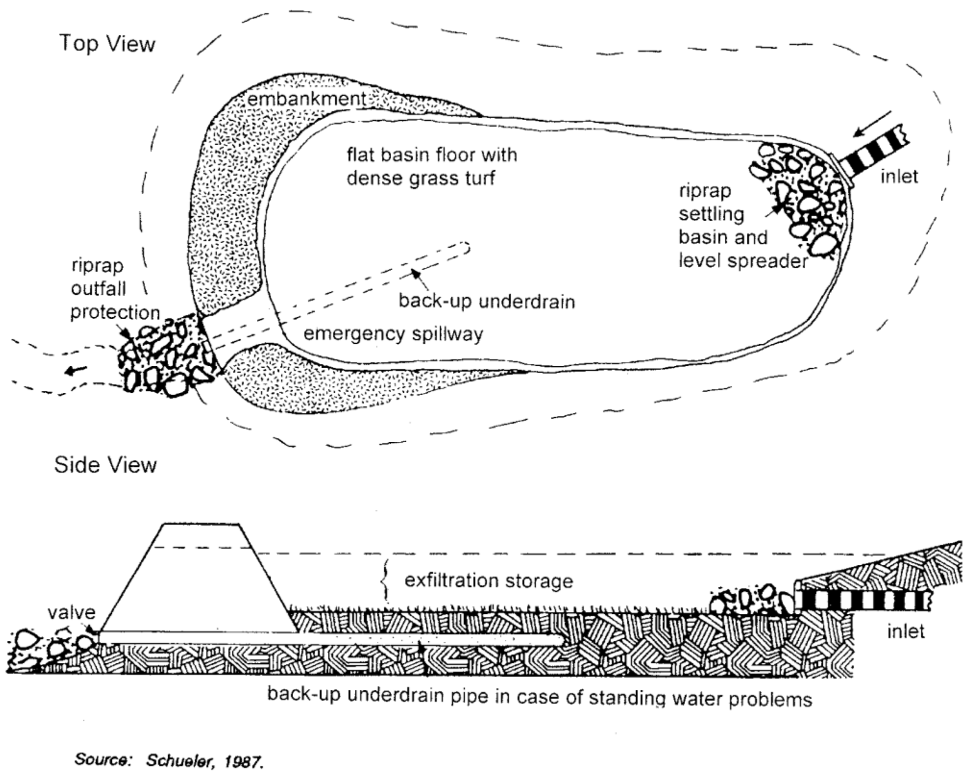

Figure 12-13 Infiltration Basins

Extended Detention Pond

Description: Conventional extended detention (ED) ponds temporarily detain a portion of stormwater runoff for up to twenty-four hours after a storm using a fixed orifice. Such extended detention allows urban pollutants to settle out. The ED ponds are normally dry between storm events and do not have any permanent standing water. Enhanced ED ponds are designed to prevent clogging and resuspension. They provide greater flexibility in achieving target detention times. Along with a detention area, they include a sediment forebay near the inlet, a micropool and/or plunge pool at the outlet, and utilize an adjustable reverse-sloped pipe as the ED.

Pollutant Removal Capability/Effectiveness

Conventional ED ponds provide moderate but variable removal of particulate pollutants, such as sediment, phosphorus and organic carbon, but provide negligible removal of soluble pollutants. Increasing detention times may result in greater removal of soluble pollutants. Pollutant removal is primarily accomplished by gravitational settling that is dependent on the detention time and the fraction of the annual runoff volume that is effectively detained in the pond (Schueler, 1987).

Feasibility

ED ponds are an adaptable BMP that can be applied to most, if not all, regions of the country. The enhanced ED pond can be utilized in most low visibility development situations, as a retrofit practice, or in combination with wetlands or permanent pools. May not be appropriate in high visibility residential or commercial settings.

Design Criteria

Figure 12-14 shows a typical diagram of extended detention ponds.

Contributing Watershed Area: In most cases, ED ponds are not practical if the watershed area is less than ten acres (Schueler, 1987).

Depth to Bedrock: If bedrock is close to the surface, high excavation costs may make ED ponds infeasible.

Depth to Water Table: If the water table is within two feet of the bottom of the ED pond, it can create problems with standing water and also indicate potential wetland status. Ground water contamination may be a problem if the soils are sufficiently porous (e.g. sandy) to allow infiltration to a high water table and stormwater runoff is expected to be contaminated.

Retrofit Capability: Frequently used for stormwater retrofits, particularly within dry stormwater management ponds and at culvert/channel intersections. Usually used in combination with a micropool, wetland or permanent pool.

Stormwater Management Capability: Frequently used in combination with two-year storm event control. Multiple outlets may be incorporated into the design to improve flexibility over a wide range of storm sizes.

Potential Benefits/Concerns

Positive Impacts:

- Extended detention is the best technique available for reducing the frequency of bank full and subbank full flooding events, and thereby is very useful in protecting downstream channels from erosion (Schueler, 1987).
- ED ponds can create both terrestrial and aquatic wildlife habitat with appropriate pondscaping and vegetation management.

- They are less hazardous than other stormwater quality ponds with deeper permanent pools.

Negative Impacts:

- ED ponds can contribute to downstream warming if pilot channels are not shaded (Galli, 1991).
- Improper site selection can create wetland, forest and habitat conflicts (Schueler, 1991).
- Poorly maintained ED ponds are not popular with adjacent residents (Adams, et. al., 1983).
- Adequate space must be available to construct the extended detention pond.
- May provide areas for insect nuisances such as mosquitos that will need control if the pond does not drain adequately between storm events.

Maintenance

Primary maintenance activities include mowing; unclogging of the ED control device; and sediment clean out in the lower stage. The ED pond has the highest routine maintenance burden of any stormwater quality pond system, due to mowing and clogging problems.

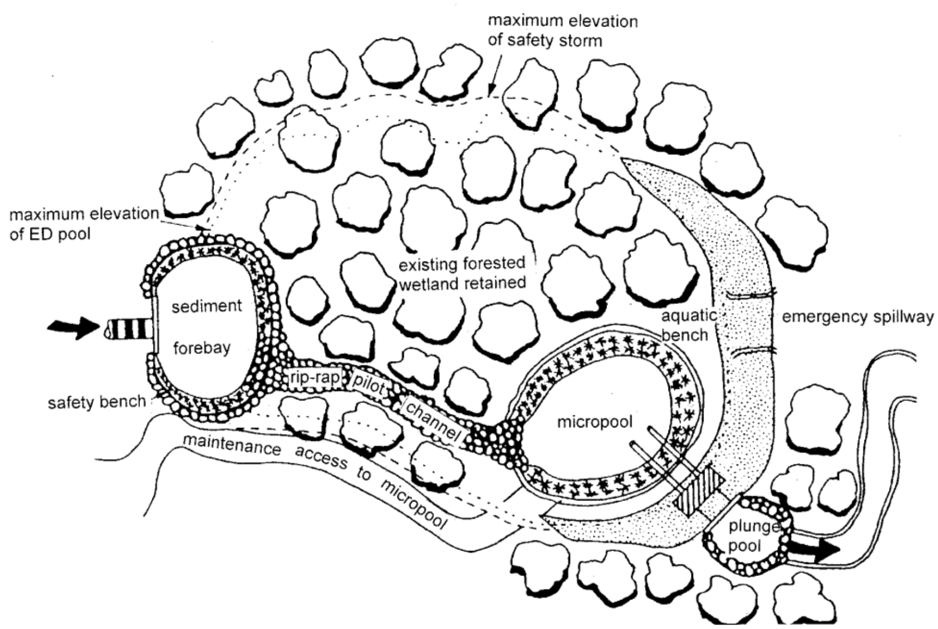

Figure 12-14 Extended Detention Pond

Inlets/Oil-Water Separators

Description: A water quality inlet (or manhole) is a three-stage underground retention system designed to remove heavy particulates and small amounts of petroleum products from stormwater runoff. They are also known as an oil/grit separator or an oil-water separator. As water flows through the three chambers, oils and grease separate either to the surface or to sediments and are skimmed off and held in the catch basin or storage tank. The stormwater then passes on to the storm sewer or into another stormwater BMP.

Pollutant Removal Capability/Effectiveness

Gravitational settling within the first two chambers can achieve partial removal of grit and sediments. Hydrocarbon removal is based on the relatively low solubility of petroleum products in water and difference between the specific gravity of water and the petroleum compounds. Oil-water separators are not designed to separate other products such as solvents, detergents or metals. Actual pollutant removal is accomplished when trapped residuals are cleaned out of the inlet (Schueler, 1987).

Current designs of water quality inlets trap coarse-grained sediments and small amounts of oil. Removal of silt, clay, nutrients, trace metals, soluble pollutants and organic matter is expected to be slight. Pollutant removal also depends on the basin volume, flow velocity, and the depth of baffles and elbows in the chamber design (NCSU, 1998). Water quality inlets may function best as a first stage in the treatment of stormwater.

Soaps, detergents, some alcohols, and other agents that emulsify oils significantly decrease the effectiveness of oil-water separators. Emulsified oil remains mixed with influent water and passes through the separator rather than being detained. Re-suspension also appears to limit long-term sediment removal. Actual removal only occurs when inlets are cleaned out. An effective clean-out and disposal schedule is essential. Under no circumstances should water quality inlets or oil-water separators be used to dispose of waste oil or other petroleum products.

Feasibility

Inlets are restricted to small, highly impervious catchments of two acres or smaller (such as gas stations, parking lots, fast food outlets, and convenience stores). Inlets (or manholes) can be adapted to all regions of the country.

Design Criteria

Water quality inlets/oil-water separators are typically proprietary structures. Manufacturers provide design guidance. A typical water quality inlet is shown in Figure 12-15.

- Most systems are applied to contributing watershed areas of two acres or less.
- The contributing areas typically are mostly or entirely impervious.
- Water quality inlets are frequently applied in dense urban areas.

Potential Benefits/Concerns

Positive Impacts:

- Trapping of floatable trash and debris.
- Potential reduction of hydrocarbon load from areas with high traffic/parking use.

Negative Impacts:

- Potential toxicity of trapped residuals.
- Possibility of pulse hydrocarbon loadings due to re-suspension during large storms.
- In some regions, it may be difficult to find environmentally acceptable disposal methods.

Maintenance

Inlets require inspections and regular clean-outs to remove accumulated sediment, oils, floatables and other pollutants. Wastes removed from these systems should be tested to determine proper disposal methods. The wastes may be hazardous; therefore, maintenance budgets should include provisions for proper disposal (NCSU, 1998). Inlets may be difficult to clean and maintain because of its enclosed, underground design (NCSU, 1998). Above ground designs do exist, though they may not be practical in cold Wyoming winters. Depending on the type of pollutants entering an oil-water separator and the configuration of the separator, they may be regulated by one or more federal, state or local programs. A brief description of possible state or federal regulation follows. Be sure to check with appropriate agencies before installing a new facility. Sludge recovered from oil-water separators may be, if hazardous, regulated by the Wyoming Department of Environmental Quality (DEQ) and/or the federal Environmental Protection Agency (EPA). Separators that include a separate tank to store waste petroleum products face additional regulations in that the waste oil tank is regulated as an underground storage tank. Existing oil-water separators that are tied to a septic system are considered class V underground injection control systems and regulated by DEQ. New connections to septic systems are no longer allowed. If you are considering an oil-water separator please contact DEQ and your local government agencies for further information.

Longevity: Longevity of water quality inlets is high. Over ninety-five percent of all inlets are operating as designed in their first five years of operation.

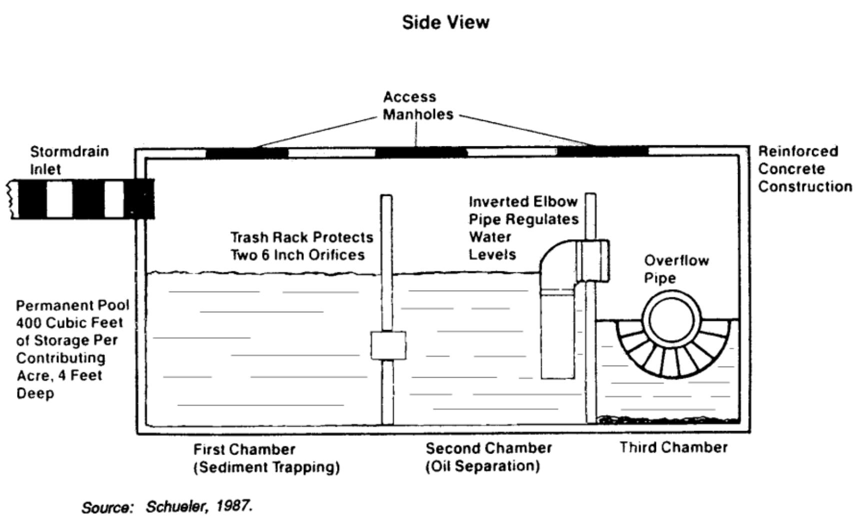

Figure 12-15 Water Quality Inlet

Retention Pond

Description: A retention pond, sometimes called a "wet pond," has a permanent pool of water with capacity above the permanent pool designed to capture and slowly release the water quality capture volume (WQCV) over a specific time period. The permanent pool is replaced, in part, with stormwater during each runoff event so stormwater runoff mixes with the permanent pool water. This allows for a reduced residence time compared to that of the extended detention basin (EDB).

Pollutant Removal Capability/Effectiveness

In general, retention ponds can be very effective in removing suspended solids, organic matter and metals through sedimentation, as well as removing soluble pollutants like dissolved metals and nutrients through biological processes.

If the retention pond includes wetlands, pollutants are removed through gravitational settling, wetland plant uptake, adsorption, physical filtration and microbial decomposition. Primary removal of stormwater pollutants occurs during the relatively long quiescent period between storms (Livingston, et. al., 1997). The degree of pollutant removal is a function of aquatic treatment volume, surface area to volume ratio, and the ratio of wetland surface area to watershed area. Additionally, longer stormwater flow paths through the wetland and longer residence times within the wetland are expected to improve pollutant removal. In western states wetland vegetation may be dormant during early spring snowmelt and rain events. However, since plant uptake is only one of several mechanisms in the removal of most pollutants, a standing crop of vegetation can still provide filtration and an area for surface removal processes (Livingston, et. al., 1997).

Feasibility

Retention ponds require groundwater or a dry-weather base flow if the permanent pool elevation is to be maintained year-round. They also require legal and physical use of water. The availability of this BMP can be limited due to water rights issues.

Design Criteria

Figure 12-16 shows a typical, enhanced constructed retention pond.

The designer should consider the overall water budget to ensure that the baseflow will exceed evaporation, evapotranspiration, and seepage losses (unless the pond is lined). High exfiltration rates can initially make it difficult to maintain a permanent pool in a new pond, but the bottom can eventually seal with fine sediment and become relatively impermeable over time. The bottom and the sides of a permanent pool should be sealed or lined if the pool is located on permeable soils, and the areas above the permanent pool should remain unsealed to promote infiltration of the stormwater above the permanent pool elevation.

Presence of Baseflow: To maintain a constant water level, it is often necessary to have a reliable dry-weather baseflow or a groundwater supply to the pond.

Surcharge Volume: Provide a surcharge volume based on a 12-hour drain time.

Permeable Soils: It is difficult to establish wet ponds at sites with sandy soils, high soil infiltration rates or high summer evapotranspiration rates.

Available Space: Because of presence of a permanent pool, retention ponds require more space than a dry, extended detention pond. If included in the design, wetlands can consume two to three times the site area compared to other stormwater quality options (in some cases, as much as five percent of total site area). The land requirements of stormwater wetlands can be sharply reduced by deepening parts of the wetland, thus extending detention times. However, side slopes along the edge of the wetland must remain gradual to maintain emergent vegetation around the wetland.

Stormwater Management Capability: In most cases, stormwater detention can be provided in stormwater wetland ponds.

Potential Benefits/Concerns

Positive Impacts:

- Retention ponds can provide an excellent urban habitat for wildlife and waterfowl, particularly if they are surrounded by a buffer and have some deeper water area (Athanas, 1986).

Negative Impacts:

- Possible impact on wetland biota from trace metal uptake (Strecker, et. al., 1990).
- Stormwater wetlands may cause warming of downstream waters (Galli, 1991).
- Construction may adversely impact existing wetland or forest areas (6, 30).
- Possible takeover by invasive aquatic nuisance plants (e.g., loosestrife, cattails and phragmites) (Stockdale, 1991).
- Bacterial contamination if waterfowl populations are dense (Wu, et. al., 1988).
- If sediment is not properly settled out before the stormwater enters the wetland, the wetland will likely become choked with sediment in a few years Provide areas for insect nuisances such as mosquitos that will need control.

Maintenance

Well-designed retention ponds should function for many years. The inclusion of a forebay or wet cell that concentrates sediment deposition in an area where it can be easily removed without disturbing the entire system is an important part of the design. Retention ponds that include wetlands may require greater maintenance in the first several years to establish the marsh. Thereafter, the maintenance burden is similar to other pond systems.

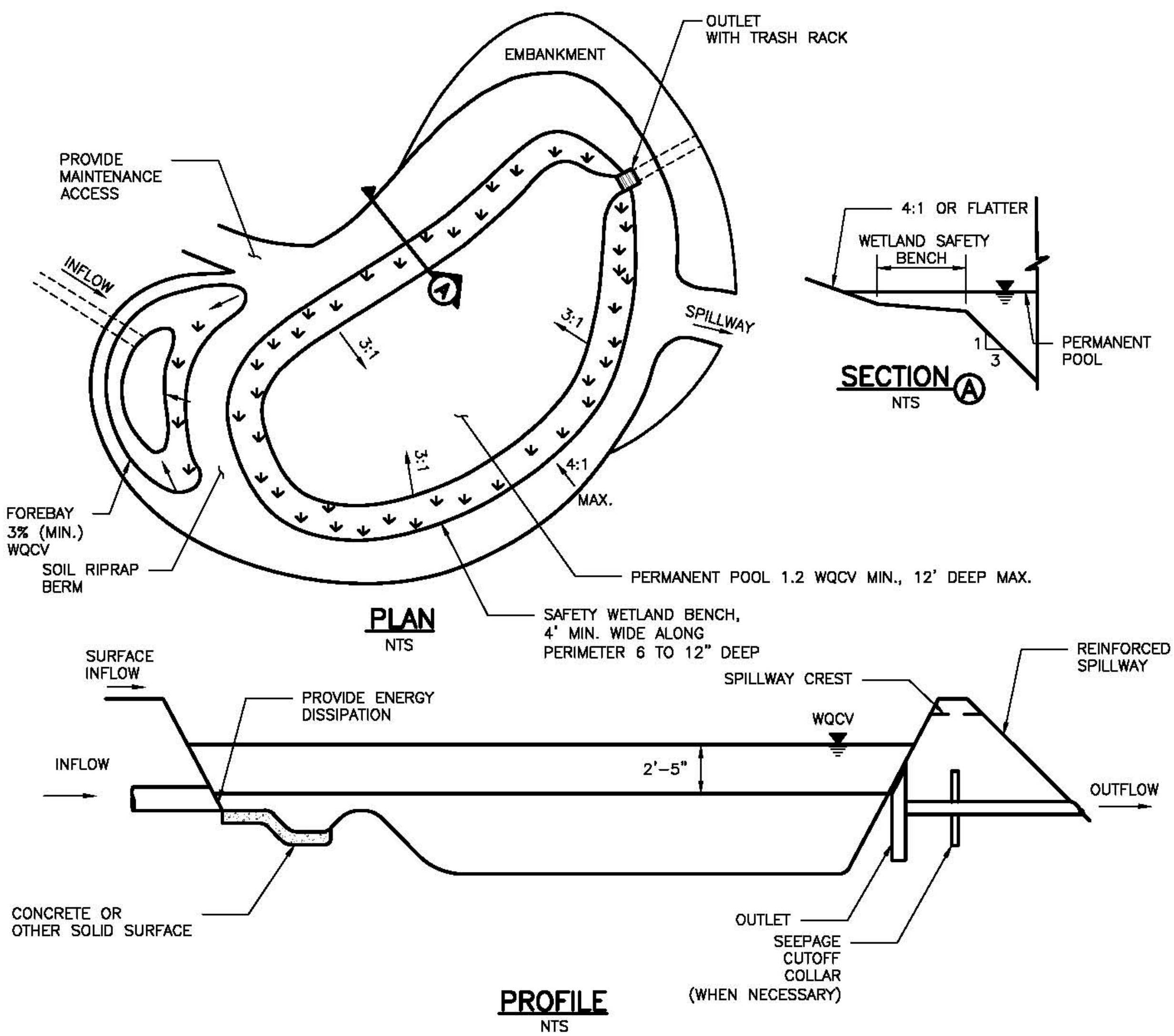

Figure 12-16 Typical Retention Pond

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Constructed Wetland Pond

Description: Conventional stormwater wetland ponds are shallow pools that create growing conditions suitable for the growth of marsh plants. These stormwater wetlands are designed to maximize pollutant removal through wetland uptake, retention and settling. Stormwater wetlands are constructed systems and typically are not located within delineated natural wetlands. In addition, stormwater wetlands differ from other artificial wetlands created to comply with mitigation requirements in that they do not replicate all the ecological functions of natural wetlands. Functional differences will depend on the design of the stormwater wetland, interactions with groundwater and surface water, and local storm climate.

Pollutant Removal Capability/Effectiveness

In general, conventional stormwater wetland ponds have a high pollutant removal capability that is generally comparable to that of conventional wet ponds. Sediment removal may be greater in well-designed stormwater wetland ponds, but phosphorus removal is more variable.

Enhanced stormwater wetlands are designed for more effective pollutant removal and species diversity. They also include design elements such as a forebay, complex microtopography, and pondscaping with multiple species of wetland trees, shrubs and plants.

Wetlands remove pollutants through gravitational settling, wetland plant uptake, adsorption, physical filtration and microbial decomposition. Primary removal of stormwater pollutants occurs during the relatively long quiescent period between storms (Livingston, et. al., 1997). The degree of pollutant removal is a function of aquatic treatment volume, surface area to volume ratio, and the ratio of wetland surface area to watershed area. Additionally, longer stormwater flow paths through the wetland and longer residence times within the wetland are expected to improve pollutant removal. In western states wetland vegetation may be dormant during early spring snowmelt and rain events. However, since plant uptake is only one of several mechanisms in the removal of most pollutants, a standing crop of vegetation can still provide filtration and an area for surface removal processes (Livingston, et. al., 1997).

Feasibility

Enhanced stormwater wetlands can be applied to most development situations where sufficient baseflow is available to maintain water elevations. Enhanced stormwater wetlands can be adapted for most regions of the country that are not excessively arid. Stormwater wetlands may not be appropriate for all areas of Wyoming. A careful review of local climate and water table conditions should be conducted before choosing this BMP.

Design Criteria

Figure 12-16 shows a typical, enhanced constructed wetland pond.

Contributing Watershed Area: Stormwater wetlands can be used in watersheds as small as five acres. However, the installation of many small wetlands increase maintenance costs (Galli, 1992). “Pocket wetlands” (generally less than 0.1 acres) have been used successfully at culvert and parking lot outlets in Minnesota (Debo and Reese, 1995).

Presence of Baseflow: To maintain a constant water level, it is often necessary to have a reliable dry-weather baseflow to the wetland or a groundwater supply.

Permeable Soils: It is difficult to establish wetlands at sites with sandy soils, high soil infiltration rates or high summer evapotranspiration rates.

Available Space: Because of their shallow depths, stormwater wetlands can consume two to three times the site area compared to other stormwater quality options (in some cases, as much as five percent of total site area). The land requirements of stormwater wetlands can be sharply reduced by deepening parts of the wetland, thus extending detention times. However, side slopes along the edge of the wetland must remain gradual to maintain emergent vegetation around the wetland.

Retrofit Capability: The addition of wetland features to older dry stormwater basins is an effective retrofit technique (Strecker, et. al., 1990). Many retrofits utilize a combination of extended detention, wetlands and a permanent pool.

Stormwater Management Capability: In most cases, stormwater detention can be provided in stormwater wetland ponds.

Potential Benefits/Concerns

Positive Impacts:

- Stormwater wetland ponds can provide an excellent urban habitat for wildlife and waterfowl, particularly if they are surrounded by a buffer and have some deeper water area (Athanas, 1986).

Negative Impacts:

- Possible impact on wetland biota from trace metal uptake (Strecker, et. al., 1990).
- Stormwater wetlands may cause warming of downstream waters (Galli, 1991).
- Construction may adversely impact existing wetland or forest areas (6, 30).
- Possible takeover by invasive aquatic nuisance plants (e.g., loosestrife, cattails and phragmites) (Stockdale, 1991).
- Bacterial contamination if waterfowl populations are dense (Wu, et. al., 1988).
- If sediment is not properly settled out before the stormwater enters the wetland, the wetland will likely become choked with sediment in a few years and provide areas for insect nuisances such as mosquitos that will need control.

Maintenance

Well-designed conventional stormwater wetlands should function for many years. The inclusion of a forebay, or wet cell that concentrates sediment deposition in an area where it can be easily removed without disturbing the entire system is an important part of the design. Stormwater wetland ponds may require greater maintenance in the first several years to establish the marsh. Thereafter, the maintenance burden is similar to other pond systems.

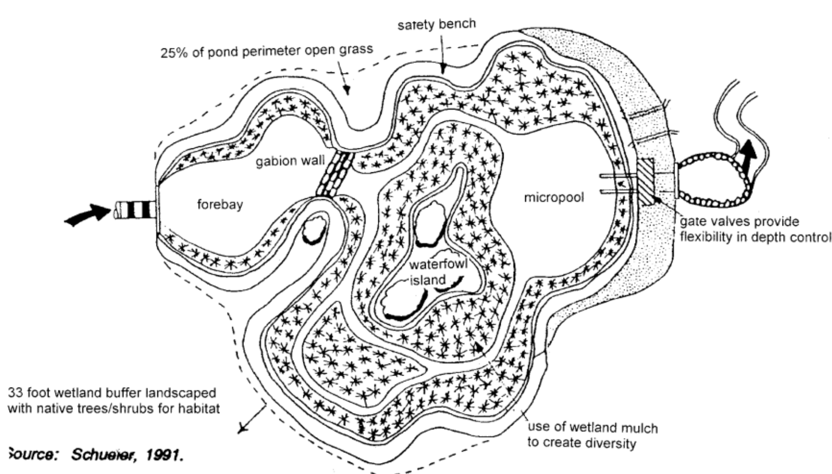

Figure 12-16 Constructed Wetland Pond

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* Date varies by chapter.

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