Storm *integrated* Water Management™

iSWM™

the comprehensive drainage resource for North Central Texas

Design Manual *for* Site Development

January 2006 Edition

North Central Texas Council of Governments

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CHAPTER

1

STORM WATER MANAGEMENT SYSTEM PLANNING AND DESIGN

Section 1.1
Storm Water Site Planning

1.1.1 Overview

1.1.1.1 Introduction

In order to most effectively and efficiently manage storm water on new development and redevelopment sites, consideration of storm water runoff needs to be fully integrated into the site planning and design process. This involves a more comprehensive approach to site planning and a thorough understanding of the physical characteristics and natural resources of the site. In addition, the management of the quantity and the quality of storm water should be addressed in an integrated approach. The purpose of this manual is to provide design guidance and a framework for incorporating effective and environmentally sensitive storm water management into the site development process and to encourage a greater uniformity in developing plans for storm water management systems that meet the following goals:

- Control conveyance of runoff within and from the site to minimize flood risk to people and properties;
- Assess discharges from the site to minimize downstream bank and channel erosion; and
- Reduce pollutants in storm water runoff to protect water quality;

When designing the storm water management system for a site, a number of questions need to be answered by the site planners and design engineers, including:

- How can the storm water management system be designed to most effectively address the quality of runoff from the site, protect against increased streambank erosion, and meet flood control objectives?
- What are the opportunities for utilizing site design practices to minimize the need for and the size of structural storm water controls?
- What are the development site constraints that preclude the use of certain structural controls?
- What structural controls are most suitable and cost-effective for the site?

1.1.1.2 Five (5) Principles of Storm Water Management Site Planning

The following Five (5) Principles of Storm Water Management Site Planning should be kept in mind in preparing an iSWM Site Plan for a development or redevelopment site:

1. **The site design should utilize an integrated approach to deal with storm water quality protection, streambank protection, and flood control requirements.**

   The storm water management infrastructure for a site should be designed to integrate drainage and flood control, water quality protection, and downstream streambank protection. Site design should be done in unison with the design and layout of storm water infrastructure to better attain storm water management goals. Together, the combination of site design practices and effective infrastructure layout and design can mitigate the worst storm water impacts of most urban developments while preserving stream integrity and aesthetic attractiveness.

2. **Storm water management practices should strive to utilize the natural drainage system and require as little maintenance as possible.**

   Almost all sites contain natural features which can be used to help manage and mitigate runoff from development. Features on a development site might include natural drainage patterns, depressions, permeable soils, wetlands, floodplains, and undisturbed vegetated areas that can be used to reduce runoff; provide infiltration, and storm water filtering of pollutants and sediment; recycle nutrients; and maximize on-site storage of storm water.

   Site design should seek to supplement the effectiveness of natural systems rather than to ignore or replace them. Further, natural systems typically require low or no maintenance, and will continue to function many years into the future.

3. **Structural storm water controls should be implemented only after all site design and nonstructural options have been exhausted.**

   Operationally, economically, and aesthetically, storm water sensitive site design and the use of natural techniques offer significant benefits over structural storm water controls. Therefore, all opportunities for utilizing these methods should be explored before implementing structural storm water controls such as wet ponds.

4. **Structural storm water solutions should attempt to be multi-purpose and be aesthetically integrated into a site's design.**

   A structural storm water facility need not be an afterthought or ugly nuisance on a development site. A parking lot, soccer field, or city plaza can serve as a temporary storage facility for storm water. In addition, water features such as ponds and lakes, when correctly designed and integrated into a site, can increase the aesthetic value of a development.

5. **"One size does not fit all" in terms of storm water management solutions.**

   Although the basic problems of storm water runoff and the need for its management remain the same, each site, project, and watershed presents different challenges and opportunities. For instance, an infill development in a highly urbanized town center or downtown area will require a much different set of storm water management solutions than a low-density residential subdivision in a largely undeveloped watershed. Therefore, local storm water management needs to take into account differences between development sites, different types of development and land use, various watershed conditions and priorities, the nature of downstream lands and waters, and community desires and preferences.

1.1.2 *integrated* Storm Water Management (iSWM) Site Plans

1.1.2.1 Introduction

To encourage and ensure that the storm water management goals are addressed and that local storm water guidelines and requirements are implemented, communities adopting this manual shall implement a formal *integrated* Storm Water Management Site Plan or iSWM Site Plan preparation, submittal, and review process that facilitates open communication and understanding between the involved parties.

An iSWM Site Plan is a comprehensive report that contains the technical information and analysis to allow a community to determine whether the storm water management system of a proposed new development or redevelopment project meets the general storm water management goals and the local storm water regulatory requirements. This section discusses the typical contents of an iSWM Site Plan, the steps for a site developer to follow in preparing an iSWM Site Plan, and the recommended review and consultation checkpoints between the local government staff and the site developer.

The procedures and guidelines for the preparation of an iSWM Site Plan should be explicitly stated in a local ordinance. The ordinance, in turn, should refer to this Manual for additional detail. Ideally, site storm water plans are developed with open lines of communication between the developer (and developer's engineer) and the plan reviewer. iSWM Site Plans are an enhanced version of traditional drainage plans, and are more than just the preparation of a document and maps. Instead, iSWM Site Plans should be thought of as a subset of the overall development process that occurs throughout the planning and development cycle of a project and then continues after construction is completed via regular inspection and maintenance of the storm water management system.

1.1.2.2 Applicability

iSWM Site Planning is applicable to land disturbing activity of 1 acre or more where total impervious area is increased by more than 5% above the current land development conditions. The Local Criteria section may provide additional definition, restrictions, or exceptions to the applicability of iSWM Site Plans based on specific watershed conditions, and may identify hotspots of concern within the local jurisdiction. (Hotspots are land uses or activities that produce higher concentrations of trace metals, hydrocarbons, or other priority pollutants. See page 5.1-8 for examples.)

New development or redevelopment in critical or sensitive areas, or as identified through a watershed study or plan, may be subject to additional performance and/or regulatory criteria. Furthermore, these sites may need to utilize certain structural controls in order to protect a special resource or address certain water quality or drainage problems identified for a drainage area or watershed.

All iSWM Site Plans shall be prepared and sealed by a Licensed Professional Engineer with a valid license from the State of Texas. The Engineer shall attest that the design was conducted in accordance with this integrated Storm Water Management Design Manual.

1.1.2.3 Contents of an iSWM Site Plan

The following elements are typical components of an iSWM Site Plan. Projects could be requested by the local jurisdiction to prepare a site plan that includes a defined subset of the elements outlined below.

1. **Existing Conditions Hydrologic Analysis**
2. **Project Description and Design Considerations**
3. **Post-Development Hydrologic Analysis**
4. **Storm Water Management System Design**
5. **Construction Storm Water Pollution Prevention Plan (SWPPP)**
6. **Landscaping Plan**

7. **Operations and Maintenance Plan**
8. **Evidence of Acquisition of Applicable Federal, State, and Local Permits**
9. **Waiver Requests**

The typical contents of each element are described further in the following sections.

1.1.3 Developer Steps to Prepare an iSWM Site Plan

1.1.3.1 Introduction

An iSWM Site Plan is a comprehensive report that contains the technical information and analysis to allow a local review authority to determine whether a proposed new development or redevelopment project meets the general storm water management goals and the local storm water regulatory requirements. The iSWM site plan shall consist of site layout mapping, narrative, supporting design calculations, and plans that sufficiently represent the proposed storm water management system (recommended scale of 1" = 50' unless otherwise specified).

This section describes the typical contents and general procedure for preparing an iSWM Site Plan. The level of detail involved in the plan will depend on the project size and the individual site and development characteristics.

Prior to beginning the steps to prepare an iSWM Site Plan, the site developer should read and understand the design approach and contents of this manual. Each of the following steps is carried forward and builds upon the previous steps to result in the Final iSWM Site Plan.

Step 1 - Consider the Five (5) Principles of Storm Water Management Site Planning

Step 2 - Review of Local Requirements

Step 3 - Perform Site Analysis and Inventory

Step 4 - Prepare Conceptual iSWM Site Plan

Step 5 - Prepare Preliminary iSWM Site Plan

Step 6 - Complete Final iSWM Site Plan

Worksheets are provided in Appendix E to assist with the development of the iSWM Site Plan.

1.1.3.2 Step 1 - Consider the Five (5) Principles of Storm Water Management Site Planning

The following principles should be kept in mind during all steps of preparing an iSWM Site Plan for a development site:

1. The site design should utilize an integrated approach to deal with storm water quality protection, streambank protection and flood control requirements.
2. Storm water management practices should strive to utilize the natural drainage system and require as little maintenance as possible.
3. Structural storm water controls should be implemented only after all site design and nonstructural options have been exhausted.
4. Structural storm water solutions should attempt to be multi-purpose and be aesthetically integrated into a site's design.
5. "One size does not fit all" in terms of storm water management solutions.

1.1.3.3 Step 2 - Review of Local Requirements

The site developer should become familiar with the local storm water management and development requirements and design criteria that apply to the site. These requirements are identified in the Local Criteria section of this manual and may include:

- Design storm frequencies
- Credits for use of *integrated* Site Design Practices
- Requirements for Water Quality Protection Volume, if applicable
- Requirements for Streambank Protection Volume
- Conveyance design criteria
- Floodplain criteria
- Buffer/setback criteria
- Wetland provisions
- Watershed-based criteria
- Erosion and sedimentation control plan
- Maintenance requirements
- Need for physical site evaluations (infiltration tests, geotechnical evaluations, etc.)
- Grading plan
- Storm Water Pollution Prevention Plan (SWPPP)

Much of this guidance can be obtained at a meeting with the local review authority and should be detailed in various local ordinances (e.g., subdivision regulations, storm water and drainage codes, etc.).

Current land use plans, comprehensive plans, zoning ordinances, road and utility plans, watershed or overlay districts, and public facility plans should all be consulted to determine the need for compliance with other local, state, and federal regulatory requirements. Guidance for applicable regulatory requirements is available in Appendix C.

Opportunities for special types of development (e.g., clustering) or special land use opportunities (e.g., conservation easements or tax incentives) should be investigated. There may also be an ability to partner with a local community for the development of greenways, or other riparian corridor or open space developments.

1.1.3.4 Step 3 - Perform Site Analysis and Inventory

Using approved field and mapping techniques, the site engineer shall collect and review information on the existing site conditions and map the following site features:

- Topography
- Drainage patterns and basins
- Intermittent and perennial streams
- Soils
- Ground cover and vegetation
- Existing development
- Existing storm water facilities

- Adjacent areas
- Property lines and easements

In addition, the site engineer shall identify and map all previously unmapped natural features such as:

- Wetlands
- Critical habitat areas
- Boundaries of wooded areas
- Floodplain boundaries
- Steep slopes
- Required buffers
- Proposed stream crossing locations
- Other required protection areas (e.g., well setbacks)

Some of this information may be available from previously performed studies or from a feasibility study. For example, some of the resource protection features may have been mapped as part of erosion and sediment control activities. Other recommended site information to map or obtain includes utilities information, seasonal groundwater levels, and geologic data.

Individual map or geographic information system (GIS) layers can be designed to facilitate an analysis of the site through what is known as map overlay, or a composite analysis. Each layer (or group of related information layers) is placed on the map in such a way as to facilitate comparison and contrast with other layers. A composite layer is often developed to show all the layers at the same time (see Figure 1.1.3-1). This composite layer can be a useful tool for defining the best buildable areas and delineating and preserving natural feature conservation areas.

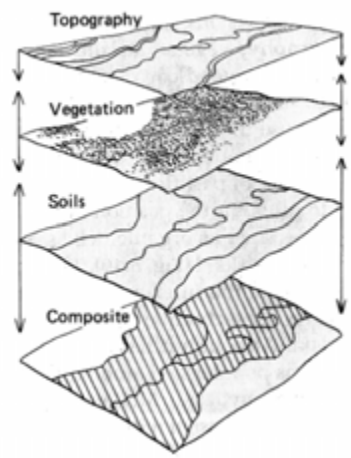

Figure 1.1.3-1
Composite Analysis
(Source: Marsh, 1983)

1.1.3.5 Step 4 - Prepare Conceptual iSWM Site Plan

Based upon the review of existing conditions and site analysis, the design engineer shall develop a Conceptual iSWM Site Plan for the project.

During the concept plan stage the site designer will perform most of the layout of the site including the conceptual storm water management system design and layout. The Conceptual iSWM Site Plan allows the design engineer to propose a potential site layout and gives the developer and local review authority a "first look" at the storm water management system for the proposed development. The Conceptual iSWM Site Plan should be submitted to the local plan reviewer before detailed preliminary site plans are developed.

The following steps shall be followed in developing the Conceptual iSWM Site Plan with the help of the Worksheet for Conceptual iSWM Site Plans found in Appendix E of this manual:

1. Use *integrated* site design practices (Section 1.3) as applicable to develop the site layout, including:
   - Preserving the natural feature conservation areas defined in the site analysis
   - Fitting the development to the terrain and minimizing land disturbance
   - Reducing impervious surface area through various techniques
   - Preserving and utilizing the natural drainage system wherever possible

2. Determine the credits for *integrated* site design (Section 1.3.4), and water quality volume reduction (Section 1.2.3.2) if applicable, to be accounted for in the design of structural and non-structural storm water controls on the site
3. Calculate conceptual estimates of the design requirements for water quality protection, streambank protection, and flood control (Section 1.2) based on the conceptual plan site layout
4. Perform screening and conceptual selection of appropriate structural storm water controls (Section1.4 and Chapter 5) and identification of potential siting locations

It is extremely important at this stage that storm water system design is integrated into the overall site design concept in order to best reduce the impacts of the development to the pre-developed drainage conditions as well as provide for the most cost-effective and environmentally sensitive approach. Using hydrologic calculations, the goal of mimicking pre-development conditions can serve a useful purpose in planning the storm water management system.

For local review purposes, following the Checklist found in Appendix E of this manual, the Conceptual iSWM Site Plan shall include site layout mapping and plans (recommended scale of 1" = 50' unless otherwise specified), which illustrate at a minimum:

Project Description

- Address and legal description of site
- Vicinity map
- Land use

Existing conditions

- Copy of applicable digital orthophotos showing proposed project boundaries
- A topographic map of existing site conditions (no greater than a 2-foot contour interval recommended) with drainage basin boundaries indicated and project boundaries shown
- Total size area (acres)
- Total impervious area as percentage (%) of total area
- Benchmarks used for site control
- Perennial and intermittent streams
- Mapping of predominant soils from USDA soil surveys
- Boundaries of existing predominant vegetation
- Location and boundaries of other natural feature protection and conservation areas such as wetlands, lakes, ponds, floodplains, stream buffers and other setbacks (e.g., drinking water well setbacks, septic setbacks, etc.)
- Location of existing roads, buildings, parking areas and other impervious surfaces
- Existing utilities (e.g., water, sewer, gas, electric) and easements
- Location of existing conveyance systems such as grass channels, swales, and storm drains
- Flow paths
- Location of floodplain/floodway limits and relationship of site to upstream and downstream properties and drainages
- Location and dimensions of existing channels, bridges or culvert crossings

Conceptual Site Layout

- Complete the iSWM Conceptual Plan Worksheet
- Hydrologic analysis to determine conceptual runoff rates, volumes, and velocities to support selection of Storm Water Controls
- Conceptual site design identifying *integrated* site design practices used

- Identification of storm water site design credits
- Identification and calculation of water quality volume reduction, if applicable
- Conceptual estimates of the three (3) storm design approach (Section 1.2) requirements
- Conceptual selection, location, and size of proposed structural storm water controls
- Conceptual limits of proposed clearing and grading
- Total proposed impervious area, as a percentage (%) of total area

The completed Conceptual iSWM Site Plan shall be submitted to the local review authority for review and comment.

1.1.3.6 Step 5 - Prepare Preliminary iSWM Site Plan

The Preliminary iSWM Site Plan ensures that requirements and criteria are being complied with and opportunities are being taken to minimize adverse impacts from the development. This step builds upon the data developed in the Conceptual iSWM Site Plan by refining and providing more detail to the concepts identified. The Worksheet for Preliminary iSWM Site Plan in Appendix E outlines the data that shall be included in the preliminary iSWM Site Plan.

The Preliminary iSWM Site Plan shall consist of maps, narrative, and supporting design calculations (hydrologic and hydraulic) for the proposed storm water management system, and shall include the following sections:

Existing Conditions Hydrologic Analysis

Provide an existing condition hydrologic analysis for storm water runoff rates, volumes, and velocities, which includes:

- Existing conditions data developed in the Conceptual iSWM Plan
- All existing storm water conveyances and structural control facilities
- Direction of flow and exits from the site
- Analysis of runoff provided by off-site areas upstream of the project site
- Methodologies, assumptions, site parameters and supporting design calculations used in analyzing the existing conditions site hydrology

Project Description and Design Considerations Provide an updated description of the project and the considerations and factors affecting the design approach that have changed between the conceptual and preliminary plans, including:

- A description of the overall project and the site plan showing facility locations, roadways, etc.
- A discussion of the applicable local criteria and how it will be integrated into the design of the project (see Local Criteria section)
- Identify the *integrated* Site Design Practices and their applicability to this site (see Section 1.3)
- If applicable, a discussion of Credits for *integrated* Site Design and their applicability to this site (see Section 1.3.4)
- A discussion of the water quality treatment techniques (pollution prevention practices) that are to be utilized on this site, if applicable
- A determination of groundwater recharge considerations, if applicable, for this site
- Identify hotspot land uses, if applicable, and how runoff will be addressed (see page 5.1-8 for examples of hotspots)

Post-Development Hydrologic Analysis

Provide a post-development hydrologic analysis using appropriate methods from Chapter 2 for storm water runoff rates, volumes, and velocities, which includes:

- A topographic map of developed site conditions (no greater than a 2-foot contour interval recommended) with the post-development basin boundaries indicated
- Total area of post-development impervious surfaces and other land cover areas for each subbasin affected by the project
- Runoff calculations for flood control and streambank protection for each subbasin, as well as any applicable water quality calculations
- Location and boundaries of proposed natural feature protection and conservation areas
- Documentation and calculations for any applicable site design credits or water quality volume reduction methods being utilized
- Methodologies, assumptions, site parameters and supporting design calculations used in analyzing the post-development conditions site hydrology
- Supporting documentation that there is existing streambank protection/reinforcement or that the planned development will provide such streambank protection downstream
- Supporting calculations for a downstream peak flow analysis to show safe passage of post-development design flows downstream. Document point downstream at which analysis ends, and how it was determined

In calculating runoff volumes and discharge rates, consideration may need to be given to any planned future upstream land use changes. Depending on the site characteristics and given local design criteria, upstream lands may need to be modeled as “existing condition” or “projected buildout/future condition” when sizing and designing on-site conveyances and storm water controls.

Storm Water Management System Design

Provide drawings and design calculations for the proposed storm water management system, including:

- A drawing or sketch of the storm water management system including the location of non-structural site design features and the placement of existing and proposed structural storm water controls. This drawing shall show design water surface elevations, storage volumes available from zero to maximum head, location of inlet and outlets, location of bypass and discharge systems, and all orifice/restrictor sizes.
- Narrative describing that appropriate and effective structural storm water controls from Chapter 5 have been selected
- Cross-section and profile drawings and design details for each of the structural storm water controls in the system. This should include supporting calculations to show that the facility is designed according to the applicable design criteria
- Hydrologic and hydraulic analysis of the storm water management system for all applicable design storms (should include stage-storage or outlet rating curves, and inflow and outflow hydrographs), refer to Chapters 2,3 and 4
- Documentation and supporting calculations to show that the storm water management system adequately meets the *integrated* Design Approach (Section 1.2)
- Drawings, design calculations and elevations for all existing and proposed storm water conveyance elements including storm water drains, pipes, culverts, catch basins, channels, swales and areas of overland flow, using guidance from Chapters 3 and 4

The completed Preliminary iSWM Site Plan shall be submitted to the local review authority for review and comment.

1.1.3.7 Step 6 - Complete Final iSWM Site Plan

The Final iSWM Site Plan adds further detail to the Preliminary iSWM Site Plan and reflects changes that are requested or required by the local review authority. The Final iSWM Site Plan, as outlined in the final iSWM Site Plan Worksheet in Appendix E, should include all of the revised elements of the Preliminary iSWM Site Plan as well as the following items:

Construction Storm Water Pollution Prevention Plan (SWPPP)

- Must contain all the elements specified in the iSWM Design Manual for Construction, local ordinances, and State regulations
- Sequence/phasing of construction and temporary stabilization measures
- Temporary structures that will be converted into permanent storm water controls

Landscaping Plan

- Arrangement of planted areas, natural areas, and other landscaped features on the site plan
- Information necessary to construct the landscaping elements shown on the plan drawings
- Descriptions and standards for the methods, materials, and vegetation that are to be used in the construction

Operations and Maintenance Plan

- Description of maintenance tasks, frequency of maintenance, responsible parties for maintenance, funding, access, and safety issues
- Reviewed and approved maintenance agreements

Evidence of Acquisition of Applicable Federal, State, and Local Permits

Description and copies of any applicable federal, state, and/or local environmental permits such as USACE Regulatory Program permits, 401 water quality certification, or construction TPDES permits. Permits must be obtained prior to or in conjunction with final plan submittal, including:

- Notice of Intent (NOI) or Construction Site Notice, as appropriate, for TPDES permits
- Permits obtained for any other storm water related development requirements (i.e. USACE Regulatory Program permits, erosion control, grading, water rights permits, TCEQ dam safety, etc.)

Waiver Requests

- Description of waiver requests

The completed Final iSWM Site Plan shall be submitted to the local review authority for final approval prior to any construction activities on the development site.

1.1.4 Local Community Plan Review Responsibilities

The iSWM Site Plans are to be reviewed by the local community. The approach recommended herein should be adopted by a community to adapt to its current plan review procedure. In most communities, part of their overall project approval process includes reviews of (1) Concept Plans, (2) Preliminary Plans, and (3) Final Plans.

The recommended approach is to incorporate the review of iSWM Site Plans into a community's current process, in an attempt to complement, not replace their overall process. Following is a brief discussion of each step with references to checklists that complement that procedure:

1. Concept Plan Stage

Review of the Conceptual iSWM Site Plan – During the concept plan stage the site designer will perform most of the layout of the site including the conceptual storm water management system design and layout. The Conceptual iSWM Site Plan allows the design engineer to propose a potential site layout and gives the developer and local review authority a "first look" at the storm water management system for the proposed development. The Conceptual iSWM Site Plan shall be submitted to and approved by the local plan reviewer before detailed preliminary site plans are developed.

It is in the Concept Plan Stage that the site developer is best able to integrate the storm water system design into the overall site design concept in order to reduce the impacts of the development, as well as provide for the most cost-effective and environmentally sensitive approach. This is done, in part, by evaluating and integrating as appropriate Credits for *integrated* Site Designs and Water Quality Protection Volume reduction. A recommended checklist for the Conceptual iSWM Site Plan Review is included in Appendix E of this Manual.

2. Preliminary Plan Stage

Review of Preliminary iSWM Site Plan - The preliminary plan ensures that local requirements and criteria are being complied with and opportunities are being taken to minimize adverse impacts from the development.

The Preliminary iSWM Site Plan shall consist of maps, narrative, and supporting design calculations (hydrologic and hydraulic) for the proposed storm water management system, and shall include the preliminary versions of the following iSWM Site Plan components:

a. Existing Conditions Hydrologic Analysis

b. Project Description and Design Considerations

c. Post-Development Hydrologic Analysis

d. Storm Water Management System Design

A recommended checklist for the Preliminary iSWM Site Plan Review is included in Appendix E of this Manual.

It shall be demonstrated that appropriate and effective storm water controls have been selected and adequately designed. The preliminary plan shall also include, among other things, street and site layout, delineation of natural feature protection and conservation areas, soils data, existing and proposed topography, relation of site to upstream drainage, limits of clearing and grading, and proposed methods to manage and maintain conservation areas (e.g., easements, maintenance agreements/responsibilities, etc.)

3. Final Plan Stage

Review Final iSWM Site Plan **-** The Final iSWM Site Plan adds further detail to the preliminary plan and reflects changes that are requested or required by the local review authority. The Final iSWM Site Plan shall include all of the revised elements from the preliminary plan as well as the following remaining items:

a. Construction Storm Water Pollution Prevention Plan (SWPPP)

b. Landscaping Plan

c. Operations and Maintenance Plan

d. Evidence of Acquisition of Applicable Federal, State, and Local Permits

e. Waiver Requests

A recommended checklist for the Final iSWM Site Plan Review is included in Appendix E of this Manual.

This process should be iterative. The reviewer should ensure that all submittal requirements have been satisfactorily addressed and permits, easements, and pertinent legal agreements (e.g., maintenance agreements, performance bond, etc.) have been obtained and/or executed.

The approved Final iSWM Site Plan shall be submitted during the platting development process to the local review authority for final approval. The iSWM Site Plan must be approved prior to the preliminary plat approval and prior to any construction activities on the development site. Approval of the Final iSWM Plan is the last major milestone in the storm water planning process. The remaining steps are to ensure the plan is installed, implemented, and maintained properly.

1.1.5 Local Community Responsibilities during Construction and Operation

After the iSWM Site Plan and the final plat have been prepared and approved, the project will move into construction and the ongoing operation of related facilities and storm water treatment areas. Again, most communities have as part of their overall process, steps to ensure that approved facilities are installed as approved and are appropriately maintained after construction. The approach recommended herein can be adopted by a community to adapt to its construction and facility review procedures. In most communities, part of their overall project process will include: (1) Construction Inspections and (2) Ongoing Maintenance Inspections.

The recommended approach is to incorporate the above steps into a community's current process, in an attempt to complement, not replace their overall process. The following steps are intended to provide a community with a review process and checklist that complement their current procedures:

1. Construction Inspections

Where possible, a pre-construction meeting shall occur before any clearing or grading is initiated on the site. This is the appropriate time to ensure that natural feature protection areas and limits of disturbance have been adequately staked and adequate erosion and sediment control measures are in place. This step ensures that the owner/developer, contractor, engineer, inspector, and plan reviewer can be sure that each party understands how the plan will be implemented on the site.

Project sites should periodically be inspected during construction by local agencies to ensure that conservation areas have been adequately protected and that storm water control and conveyance facilities are being constructed as designed. Inspection frequency may vary with regard to site size and location; however, monthly inspections are a minimum target. In addition, it is recommended that some inspections occur after larger storm events (e.g., 0.5 inches and greater to assure compliance with the TPDES Construction General Permit). The inspection process can prevent later problems that result in penalties and added cost to developers.

An added benefit of a formalized and regular inspection process is that it should help to motivate contractors to internalize regular maintenance of sediment controls as part of the daily construction operations and not disturb water quality area "set-asides." If necessary, a community can consider implementing a penalty system whereby things such as stop work orders could be issued.

A final inspection is needed to ensure that the construction conforms to the intent of the approved design. Prior to accepting the infrastructure components, issuing an occupancy permit, and releasing any applicable bonds, the review authority should ensure that: (1) temporary erosion control measures have been removed; (2) storm water controls are unobstructed and in good working order;

(3) permanent vegetative cover has been established in exposed areas; (4) any damage to natural feature protection and conservation areas has been mitigated; (5) conservation areas and buffers have been adequately marked or signed; and (6) any other applicable conditions have been met.

Record drawings of the structural storm water controls, drainage facilities, and other infrastructure components should also be acquired by the community, as they are important in the long-term maintenance of the facilities. The review authority should keep copies of the drawings and associated documents and develop a local storm water control inventory and data storage system. With geographic information systems (GIS) becoming more widely used, much of this data can be stored electronically.

2. Ongoing Maintenance Inspections

Ongoing inspection and maintenance of a project site's storm water management system is often the weakest component of storm water plans. It needs to be clearly detailed in the iSWM Site Plan which entity has responsibility for operation and maintenance of all structural storm water controls and drainage facilities. Often, the responsibility for maintenance is transferred from the developer and contractor to the owner. Communication about this important responsibility is usually inadequate. Therefore, communities may need to consider ways to notify property owners of their responsibilities. For example, notification can be made through a legal disclosure upon sale or transfer of property or public outreach programs may be instituted to describe the purpose and value of maintenance.

Ideally, preparation of maintenance plans should be a requirement of the iSWM Site Plan preparation and review process. A maintenance plan should outline the scope of activities, schedule, costs, funding source, and responsible parties. Vegetation, sediment management, access, and safety issues should also be addressed. In addition, the plan should address the testing and disposal of sediments that will likely be necessary and the ultimate replacement of structures as needed.

Annual inspections of storm water management facilities should be conducted by an appropriate local agency. Where chronic or severe problems exist, the local government should have the authority to remedy the situation and charge the responsible party for the cost of the work. This authority should be well established in an ordinance.

1.1.6 iSWM Site Plan Design Tools

There are several design tools that can be used by the developer, planner, and engineer in the development of an iSWM Site Plan for a specific project. The tools include the following, which are discussed in more detail in subsequent sections of this Manual:

***integrated* Planning and Design Approach** (Section 1.2)

- Design requirements to achieve water quality protection, streambank protection, and flood control goals

***integrated* Site Design Practices** (Section 1.3)

- Nonstructural approaches to be used during site planning

***integrated* Storm Water Controls** (Section 1.4)

- Controls to remove pollutants, regulate discharge, and/or convey storm water

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Section 1.2
integrated Planning and Design Approach

1.2.1 Introduction

This section presents an integrated approach for meeting the storm water runoff quality and quantity management goals by addressing the key adverse impacts of development on storm water runoff. The purpose is to provide guidance for designing a comprehensive storm water management system as part of the iSWM Site Plan to:

- Remove pollutants in storm water runoff to protect water quality;
- Assess discharge from the site to minimize downstream bank and channel erosion; and
- Control conveyance of runoff within and from the site to minimize flood risk to people and property.

The *integrated* Design Approach is a coordinated set of design standards that allow the site engineer to design and size storm water controls to address these goals. Each of the *integrated* Design Steps should be used in conjunction with the others to address the overall storm water impacts from a development site. When used as a set, the *integrated* Design Approach controls the entire range of hydrologic events, from the smallest runoff-producing rainfalls up to the 100-year, 24-hour storm. Through the *integrated* Design Approach, each community receives standardized options while retaining the flexibility to define their own program. The Local Criteria section of this manual specifies the options allowed and/or required by the community.

The design approach for each of the goals above is summarized in Table 1.2.1-1 below:

Table 1.2.1-1 Steps for *integrated* Design Approach for Storm Water Control and Impact Mitigation

Steps	Approach
Step 1: Downstream Assessment	Conduct a downstream assessment to the point at which the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system. The assessment shall analyze downstream impacts from a development for three (3) storm events based on Local Criteria: (1) a "Streambank Protection" storm, either the 1- or 2-year, 24-hour event, (2) a "Conveyance" storm, either the 5-, 10-, or 25-year, 24-hour event, and (3) the "100-year" storm, a 100-year 24-hour storm event.
Step 2: Water Quality Protection	Achieve by conserving natural features, reducing impervious cover, and using the natural drainage system by applying the *integrated* Site Design Practices; as approved by the local jurisdiction. For enhanced water quality protection use one or both of the following options: (1) If required by the Local Criteria, treat the Water Quality Protection Volume (WQ_V) by reducing total suspended solids from the development site for runoff resulting from rainfalls of up to 1.5 inches (85th percentile storm); (2) Assist in implementing off-site community storm water pollution prevention programs/activities as designated in an approved storm water master plan or TPDES Storm Water permit.

Table 1.2.1-1 Steps for *integrated* Design Approach for Storm Water Control and Impact Mitigation	
Step 3: Streambank Protection	Provide streambank protection from erosion due to increased storm water volumes and velocities caused by development using one or more of the following options: (1) Determine acceptable downstream conditions; (2) Reinforce/stabilize downstream conditions; (3) Install storm water controls to maintain existing downstream conditions; (4) Provide on-site controlled release of the 1-year, 24-hour storm event over a period of 24 hours (Streambank Protection Volume, SP_V).
Step 4: Flood Control	Flood impact reduction may be achieved by a combination of on-site control, downstream protection, floodplain management, and/or other mitigation measures. Onsite: Minimize localized site flooding of streets, sidewalks, and properties by a combination of on-site storm water controls and conveyance systems. These systems will be designed for the "Streambank Protection" and "Conveyance" storm event frequencies. Depending upon their location, function, and the requirements of the local jurisdiction, the full build-out "100-year" storm event is to be conveyed on-site such that no resulting habitable structural flooding occurs. Downstream: Based on the downstream assessment, manage downstream flood impacts caused by the increase of storm water discharges from the development using one or more of following options: (1) Determine acceptable downstream conditions; (2) Provide adequate downstream conveyance systems, (3) Install storm water controls on-site to maintain existing downstream conditions; (4) In lieu of a downstream assessment, maintain existing on-site runoff conditions.

Figure 1.2.1-1 graphically illustrates the relative volume requirements of each of the *integrated* Design Steps and demonstrates that the pieces typically overlay one another. If the downstream assessment for flood control indicated upstream detention was needed to limit the discharge from a development, the volume requirement to achieve the downstream flood control requirement could also contain the volume needed to provide for Streambank Protection and, if required, Water Quality Protection. The appropriate type of detention facility could be designed with outlet controls to address each of the steps of the Design Approach. Obviously, detention may not be required in all situations, but consideration of site design practices and storm water controls that work together to meet all the requirements is what is important. The following sections describe the *integrated* Design Approach in more detail.

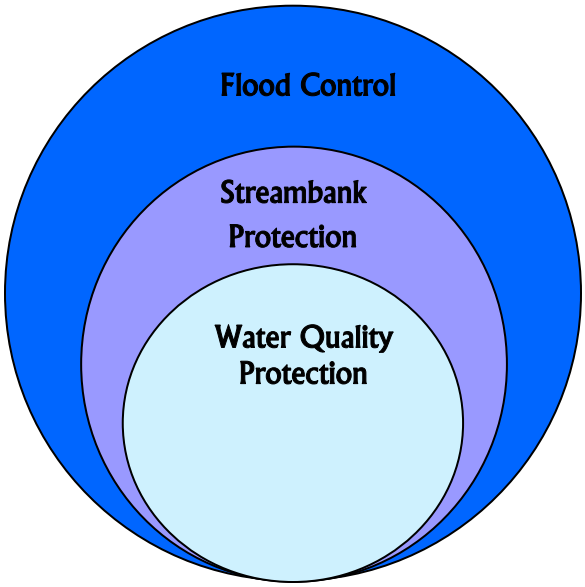

Figure 1.2.1-1 Representation of the *integrated* Design Approach

1.2.2 Downstream Assessment

As part of the iSWM Site Plan development, the downstream impacts of development must be carefully evaluated. The purpose of the downstream assessment is to protect downstream properties from increased flooding and downstream channels from increased erosion potential due to upstream development. The importance of the downstream assessment is particularly evident for larger sites or developments that have the potential to dramatically impact downstream areas. The cumulative effect of smaller sites, however, can be just as dramatic and, as such, following the *integrated* Design Approach is just as important for the smaller sites as it is for the larger sites.

The assessment should extend from the outfall of a proposed development to a point downstream where the discharge from a proposed development no longer has a significant impact on the receiving stream or storm drainage system. The assessment should be a part of the concept, preliminary, and final iSWM site plans, and should include the following properties:

- Hydrologic analysis of the pre- and post-development on-site conditions
- Drainage path which defines extent of the analysis.
- Capacity analysis of all existing constraint points along the drainage path, such as existing floodplain developments, underground storm drainage systems culverts, bridges, tributary confluences, or channels
- Offsite undeveloped areas are considered as “full build-out” for both the pre- and post-development analyses
- Evaluation of peak discharges and velocities for three (3) 24-hour storm events
  - Small-frequency storm for “Streambank Protection”, either the 1- or 2-year event
  - A “Conveyance” storm of either the 5-, 10-, or 25-year event
  - A “100-year” storm event
- Separate analysis for each major outfall from the proposed development

Once the analysis is complete, the designer should ask the following three questions at each determined junction downstream:

- Are the post-development discharges greater than the pre-development discharges?
- Are the post-development velocities greater than the pre-development velocities?
- Are the post-development velocities greater than the velocities allowed for the receiving system?

These questions should be answered for each of the three storm events. The answers to these questions will determine the necessity, type, and size of non-structural and structural controls to be placed on-site or downstream of the proposed development. Section 2.1 gives additional guidance on calculating the discharges and velocities, as well as determining the downstream extent of the assessment.

1.2.3 Water Quality Protection

iSWM requires the use of *integrated* Site Design Practices as the primary means to protect the water quality of our streams, lakes, and rivers from the negative impacts of storm water runoff from development. A community should provide adequate water quality protection for development sites by specifying in their Local Criteria the acceptable *integrated* Site Design Practices for the local community. Enhanced water quality protection shall only be required as identified by the Local Criteria section of this manual.

<u>Use of *integrated* Site Design Practices</u>

Through the consideration and use of *integrated* Site Design practices, as discussed in Section 1.3, natural drainage and treatment systems can be preserved. With conservation of natural features, reduced imperviousness, and the use of the natural drainage system, the generation of storm water runoff

and pollutants from the site is reduced.

Enhanced water quality protection can be achieved by use of one or both of the following two options:

Option 1: Treat the Water Quality Protection Volume

A municipality may identify specific watersheds with documented poor water quality and require design enhancements as a part of the on-site controls to address water quality protection. Therefore, using the Water Quality Protection Volume as required by the Local Criteria, storm water runoff generated from sites can be treated using a variety of on-site structural and nonstructural techniques with the goal of removing a target percentage of the average annual total suspended solids.

A system has been developed by which the Water Quality Protection Volume can be reduced, thus requiring less structural control. This is accomplished through the use of certain reduction methods, where affected areas can be deducted from the site area ("A") in the formula, thereby reducing the amount of runoff to be treated ("WQ_v"). For more information on the Water Quality Volume Reduction Methods see Section 1.2.3.2.

Option 2: Assist with Off-site Pollution Prevention Activities/Programs

Some communities may implement pollution prevention programs/activities in certain areas to remove pollutants from the runoff after it has been discharged from the site. This may be especially true in intensely urbanized areas facing site redevelopment where many of the BMP criteria would be difficult to apply. These programs will be identified in the local jurisdiction's approved TPDES storm water permit. In lieu of on-site treatment, the developer may be requested to simply assist with the implementation of these off-site pollution prevention programs/activities.

Through iSWM, each community receives standardized tools while retaining the flexibility to define their own program. The Local Criteria section of this manual allows for this flexibility.

1.2.3.1 Water Quality Protection Volume

Hydrologic studies show smaller, frequently occurring storms account for the majority of rainfall events. Consequently, the runoff from the many smaller storms also accounts for a major portion of the annual pollutant loadings. By treating these frequently-occurring, smaller rainfall events and the initial portion of the storm water runoff from larger events, it is possible to effectively mitigate the water quality impacts from a developed area.

Studies have shown the 85th percentile storm event (i.e., the storm event that is greater than 85% of the storms that occur) is a reasonable target event to address the vast majority of smaller, pollutant-loaded storms. Based on a rainfall analysis, 1.5 inches of rainfall has been identified as the average depth corresponding to the 85th percentile storm for the NCTCOG region. The runoff from these 1.5 inches of rainfall is referred to as the Water Quality Protection Volume (WQ_v). Thus, a storm water management system designed for the WQ_v will treat the runoff from all storm events of 1.5 inches or less, as well as a portion of the runoff for all larger storm events. The Water Quality Protection Volume is directly related to the amount of impervious cover and is calculated using the formula below:

$$\mathbf{WQ_V = \frac{1.5R_V A}{12}}$$

where:

WQ_v = Water Quality Protection Volume (in acre-feet)

R_v = 0.05 + 0.009(I) where I is percent impervious cover

A = site area in acres remaining after reduction

Determining the Water Quality Protection Volume (WQ_v)

- *Measuring Impervious Area*: The area of impervious cover can be taken directly off a set of plans or appropriate mapping. Where this is impractical, NRCS TR-55 land use/impervious cover relationships can be used to estimate impervious cover. I is expressed as a percent value not a fraction (e.g., I = 30 for 30% impervious cover)
- *Multiple Drainage Areas*: When a development project contains or is divided into multiple outfalls, WQ_v should be calculated and addressed separately for each outfall.
- *Water Quality Volume Reduction*: The use of certain *integrated* site design practices may allow the WQ_v to be reduced. These volume reduction methods are described in Section 1.2.3.3.
- *Determining the Peak Discharge for the Water Quality Storm:* When designing off-line structural control facilities, the peak discharge of the water quality storm (Q_{wq}) can be determined using the method provided in Section 2.1.10.2.
- *Extended Detention of the Water Quality Volume:* The water quality treatment requirement can be met by providing a 24-hour drawdown of a portion of WQ_v in a storm water pond or wetland system (as described in Chapter 5). Referred to as water quality ED (extended detention), it is different than providing extended detention of the 1-year storm for the streambank protection volume (SP_v). The ED portion of the WQ_v may be included when routing the SP_v.
- *Permanent Pool:* Wet ponds and wetlands will have permanent pools, the volume of which may be used to account for up to 50% of the WQ_v.
- WQ_v can be expressed in cubic feet by multiplying by 43,560. WQ_v can also be expressed in watershed-inches by removing the area (A) and the "12" in the denominator.

This approach to control pollution from storm water runoff treats the WQ_v from a site to reduce a target percentage of post-development total suspended solids (TSS). TSS was chosen as the representative storm water pollutant for measuring treatment effectiveness for several reasons:

- The measurement standard of using TSS as an "indicator" pollutant is well established.
- Suspended sediment and turbidity, as well as other pollutants of concern adhere to suspended solids, and are a major source of water quality impairment due to urban development in the region's watersheds.
- A large fraction of many other pollutants of concern are removed either along with TSS, or at rates proportional to the TSS removal.

Even though TSS is a good indicator for many storm water pollutants, there are special cases that warrant further consideration including:

- The removal performance for pollutants that are soluble or that cannot be removed by settling must be specifically designed for. For pollutants of specific concern, individual analyses of specific pollutant sources should be performed and the appropriate removal mechanisms implemented.
- Runoff, which is atypical in terms of normal TSS concentrations, will be treated to a higher or lesser degree. For example, treatment of highly turbid waters would attain a higher removal percentage but still may not attain acceptable water quality without additional controls or a higher level of BMP maintenance.
- Bed and bank-material sediment loads not accurately measured by the TSS standard are also typically removed using this approach.
- Site, stream, or watershed specific criteria, different from the TSS standard, may be developed through a state or federal regulatory program necessitating a tailored approach to pollution prevention.

1.2.3.2 Water Quality Volume Reduction Methods

A set of storm water "volume reduction methods" is presented to provide developers and site designers an incentive to implement site designs that can reduce the volume of storm water runoff and minimize the pollutant loads from a site. The reduction directly translates into cost savings to the developer by reducing the size of structural storm water control and conveyance facilities.

The basic premise of the system is to recognize the water quality benefits of certain site design practices by allowing for a reduction in the water quality protection volume (WQ_v). If a developer incorporates one or more of the methods in the design of the site, the requirement for capture and treatment of the water quality protection volume will be reduced.

The methods by which the water quality volume can be reduced are listed in Table 1.2.3-1. Site-specific conditions will determine the applicability of each method. For example, the stream buffer reduction cannot be taken on upland sites that do not contain perennial or intermittent streams. Perennial streams flow 365 days a year in a normal year. Intermittent streams have short or lengthy periods of time when there is no flow in a normal year.

Table 1.2.3-1 Methods to Reduce the Water Quality Volume

Practice	Description
Natural area conservation	Undisturbed natural areas are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics.
Stream buffers	Storm water runoff is treated by directing sheet flow runoff through a naturally vegetated or forested buffer as overland flow.
Use of vegetated channels	Vegetated channels are used to provide storm water treatment.
Overland flow filtration/infiltration zones	Overland flow filtration/infiltration zones are incorporated into the site design to receive runoff from rooftops and other small impervious areas.
Environmentally sensitive large lot subdivisions	A group of site design techniques are applied to low and very low density residential development.

Site designers are encouraged to use as many volume reduction methods as they can on a site. Greater reductions in storm water storage volumes can be achieved when many methods are combined (e.g., disconnecting rooftops and protecting natural conservation areas). However, volume reduction cannot be claimed twice for an identical area of the site (e.g. claiming a reduction for stream buffers and disconnecting rooftops over the same site area).

Due to local safety codes, soil conditions, and topography, some of these volume reduction methods may be restricted. Designers are encouraged to consult with the appropriate approval authority to ensure if and when a reduction is applicable and to determine restrictions on non-structural strategies.

The methods by which the water quality volume can be reduced are detailed below. For each volume reduction method, there is a minimum set of criteria and requirements that identify the conditions or circumstances under which the reduction may be applied. The intent of the suggested numeric conditions (e.g., flow length, contributing area, etc.) is to avoid situations that could lead to a volume reduction being granted without the corresponding reduction in pollution attributable to an effective site design modification.

Volume Reduction Method #1: Natural Area Conservation

A water quality volume reduction can be taken when undisturbed natural areas are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics. Under this method, a designer would be able to subtract the conservation areas from the total site area when computing the water quality protection volume. An added benefit is that the post-development peak discharges will be smaller, and hence, water quantity control volumes will be reduced due to lower post-development curve numbers or rational formula "C" values.

Rule: Subtract conservation areas from total site area when computing water quality protection volume requirements.

Criteria:

- Conservation area cannot be disturbed during project construction and must be protected from sediment deposition.
- Shall be protected by limits of disturbance clearly shown on all construction drawings
- Shall be located within an acceptable conservation easement instrument that ensures perpetual protection of the proposed area. The easement must clearly specify how the natural area vegetation shall be managed and boundaries will be marked [Note: managed turf (e.g., playgrounds, regularly maintained open areas) is not an acceptable form of vegetation management]
- Shall have a minimum contiguous area requirement of 10,000 square feet
- R_v is kept constant when calculating WQ_v
- Must be forested or have a stable, natural ground cover.

Example:

Residential Subdivision
Area = 38 acres
Natural Conservation Area = 7 acres
Impervious Area = 13.8 acres

$R_v = 0.05 + 0.009\ (I) = 0.05 + 0.009\ (36.3\%) = 0.38$

Reduction:

7.0 acres in natural conservation area

New drainage area = 38 – 7 = 31 acres

Before reduction:
$WQ_v = (1.5)(0.38)(38)/12 = 1.81$ ac-ft

With reduction:
$WQ_v = (1.5)(0.38)(31)/12 = 1.47$ ac-ft

(19% reduction in water quality protection volume)

Volume Reduction Method #2: Stream Buffers

This reduction can be taken when a stream buffer effectively treats storm water runoff. Effective treatment constitutes treating runoff through overland flow in a naturally vegetated or forested buffer. Under the proposed method, a designer would be able to subtract areas draining via overland flow to the buffer from total site area when computing water quality protection volume requirements. In addition, the volume of runoff draining to the buffer can be subtracted from the streambank protection volume. The design of the stream buffer treatment system must use appropriate methods for conveying flows above the annual recurrence (1-yr storm) event.

Rule: Subtract areas draining via overland flow to the buffer from total site area when computing water quality protection volume requirements.

Criteria:

- The minimum undisturbed buffer width shall be 50 feet
- The maximum contributing length shall be 150 feet for pervious surfaces and 75 feet for impervious surfaces
- The average contributing slope shall be 3% maximum unless a flow spreader is used
- Runoff shall enter the buffer as overland sheet flow. A flow spreader can be installed to ensure this
- Buffers shall remain as naturally vegetated or forested areas and will require only routine debris removal or erosion repairs
- R_v is kept constant when calculating WQ_v
- Not applicable if overland flow filtration/groundwater recharge reduction is already being taken

Example:

Residential Subdivision
Area = 38 acres
Impervious Area = 13.8 acres
Area Draining to Buffer = 5 acres

$R_v = 0.05 + 0.009\ (I) = 0.05 + 0.009\ (36.3\%) = 0.38$

Reduction:

5.0 acres draining to buffer

New drainage area = 38 – 5 = 33 acres

Before reduction:
WQ_v = (1.5)(0.38)(38)/12 = 1.81 ac-ft

With reduction:
WQ_v = (1.5)(0.38)(33)/12 = 1.57 ac-ft

(13% reduction in water quality protection volume)

Volume Reduction Method #3: Enhanced Swales

This reduction may be taken when enhanced swales are used for water quality protection. Under the proposed method, a designer would be able to subtract the areas draining to an enhanced swale from total site area when computing water quality protection volume requirements. An enhanced swale can fully meet the water quality protection volume requirements for certain kinds of low-density residential development (see Volume Reduction Method #5). An added benefit is the post-development peak discharges will likely be lower due to a longer time of concentration for the site.

Rule: Subtract the areas draining to an enhanced swale from total site area when computing water quality protection volume requirements.

Criteria:

- This method is typically only applicable to moderate or low density residential land uses (3 dwelling units per acre maximum)
- The maximum flow velocity for water quality design storm shall be less than or equal to 1.0 feet per second
- The minimum residence time for the water quality storm shall be 5 minutes
- The bottom width shall be a maximum of 6 feet. If a larger channel is needed use of a compound cross section is required
- The side slopes shall be 3:1 (horizontal:vertical) or flatter
- The channel slope shall be 3 percent or less
- R_v is kept constant when calculating WQ_v

Example:

Residential Subdivision
Area = 38 acres
Impervious Area = 13.8 acres

$R_v = 0.05 + 0.009\ (I) = 0.05 + 0.009\ (36.3\%) = 0.38$

Reduction:

12.5 acres meet enhanced swale criteria

New drainage area = 38 – 12.5 = 25.5 acres

Before reduction:
$WQ_v = (1.5)(0.38)(38)/12 = 1.81$ ac-ft

With reduction:
$WQ_v = (1.5)(0.38)(25.5)/12 = 1.21$ ac-ft

(33% reduction in water quality protection volume)

Volume Reduction Method #4: Overland Flow Filtration/Groundwater Recharge Zones

This reduction can be taken when "overland flow filtration/infiltration zones" are incorporated into the site design to receive runoff from rooftops or other small impervious areas (e.g., driveways, small parking lots, etc). This can be achieved by grading the site to promote overland vegetative filtering or by providing infiltration or "rain garden" areas. If impervious areas are adequately disconnected, they can be deducted from total site area when computing the water quality protection volume requirements. An added benefit will be that the post-development peak discharges will likely be lower due to a longer time of concentration for the site.

Rule: If impervious areas are adequately disconnected, they can be deducted from total site area when computing the water quality protection volume requirements.

Criteria:

- Relatively permeable soils (hydrologic soil groups A and B) should be present
- Runoff shall not come from a designated hotspot
- The maximum contributing impervious flow path length shall be 75 feet
- Downspouts shall be at least 10 feet away from the nearest impervious surface to discourage "re-connections"
- The disconnection shall drain continuously through a vegetated channel, swale, or filter strip to the property line or structural storm water control
- The length of the "disconnection" shall be equal to or greater than the contributing length
- The entire vegetative "disconnection" shall be on a slope less than or equal to 3 percent
- The surface imperviousness area to any one discharge location shall not exceed 5,000 square feet
- For those areas draining directly to a buffer, reduction can be obtained from either overland flow filtration *-or-* stream buffers (See Method #2)
- R_v is kept constant when calculating WQ_v

Example:

Site Area = 3.0 acres
Impervious Area = 1.9 acres (or 63.3% impervious cover)
"Disconnected" Impervious Area = 0.5 acres

$R_v = 0.05 + 0.009\ (I) = 0.05 + 0.009\ (63.3\%) = 0.62$

Reduction:

0.5 acres of surface imperviousness hydrologically disconnected

New drainage area = 3 – 0.5 = 2.5 acres

Before reduction:
WQ_v = (1.5)(0.62)(3)/12 = 0.23 ac-ft

With reduction:
WQ_v = (1.5)(0.62)(2.5)/12 = 0.19 ac-ft

(17% reduction in water quality protection volume)

Volume Reduction Method #5: Environmentally Sensitive Large Lot Subdivisions

This reduction can be taken when a group of environmental site design techniques are applied to low and very low density residential development (e.g., 1 dwelling unit per 2 acres [du/ac] or lower). The use of this method can eliminate the need for structural storm water controls to treat water quality protection volume requirements. This method is targeted towards large lot subdivisions and will likely have limited application.

Rule: Targeted towards large lot subdivisions (e.g. 2 acre lots and greater). The requirement for structural practices to treat the water quality protection volume shall be waived.

Criteria:

For Single Lot Development:

- Total site impervious cover is less than 15%
- Lot size shall be at least two acres
- Rooftop runoff is disconnected in accordance with the criteria in Method #4
- Grass channels are used to convey runoff versus curb and gutter

For Multiple Lots:

- Total impervious cover footprint shall be less than 15% of the area
- Lot areas should be at least 2 acres, unless clustering is implemented. Open space developments should have a minimum of 25% of the site protected as natural conservation areas and shall be at least a half-acre average individual lot size
- Grass channels should be used to convey runoff versus curb and gutter (see Method #3)
- Overland flow filtration/infiltration zones should be established (see Method #4)

1.2.4 Streambank Protection

The increase in the frequency and duration of bankfull flow conditions in stream channels due to urban development is the primary cause of accelerated streambank erosion and the widening and downcutting of stream channels. Therefore, streambank protection criterion applies to all development sites for which there is an increase in the natural flows to downstream feeder streams, channels, ditches, and small streams.

There are four options by which a community can provide adequate streambank protection downstream of a proposed development. The local jurisdiction should specify in their Local Criteria which of these options are acceptable, as well as any other alternatives for streambank protection. If on-site or downstream improvements are required for streambank protection, easements or right-of-entry agreements may need to be obtained in accordance with the Local Criteria.

Option 1: Determine Acceptable Downstream Conditions

The developer should first determine if existing downstream streambank protection is adequate to convey storm water velocities for post-development conditions. This is accomplished by first obtaining post-developed velocities for the “Streambank Protection” storm event from the downstream assessment, as described in Section 1.2.2. These velocities are then compared to the allowable velocity of the downstream receiving system. Allowable velocities can be found in Chapter 4 in Tables 4.4-2 and 4.4-3. If the downstream system is designed to handle the increase in velocity, the developer should provide all supporting calculations and/or documentation to show that the stream integrity will not be compromised.

Option 2: Reinforce/Stabilize Downstream Conditions

If the increased velocities are higher than the allowable velocity of the downstream receiving system, then the developer may choose to reinforce/stabilize the downstream conveyance system. The proposed modifications must be designed so that the downstream post-development velocities (for the 3 storm events described in Section 1.2.2) are less than or equal to either the allowable velocity of the downstream receiving system or the pre-development velocities, whichever is higher. The developer must provide supporting calculations and/or documentation that the downstream velocities do not exceed the allowable range once the downstream modifications are installed. (See Tables 4.4-2 and 4.4-3 for allowable velocities.)

Option 3: Install Storm Water Controls On-site to Maintain Existing Downstream Conditions

The developer may also choose to use on-site controls to keep downstream post-development discharges at or below allowable velocity limits described in Option 2. The developer must provide supporting calculations and/or documentation that the on-site controls will be designed such that downstream velocities for the three (3) storm events described in Section 1.2.2 are within an allowable range once the controls are installed.

Option 4: Provide On-site Controlled Release of the Streambank Protection Volume

Another approach to streambank protection is to specify that 24 hours of extended detention be provided for on-site, post-developed runoff generated by the 1-year, 24-hour rainfall event to protect downstream channels. The required volume for extended detention is referred to as the Streambank Protection Volume (denoted SP_v). The reduction in the frequency and duration of bankfull flows through the controlled release provided by extended detention of the SP_v will reduce the bank scour rate and severity.

Determining the Streambank Protection Volume (SP_v)

- *SP_v Calculation Methods:* Several methods can be used to calculate the SP_v storage volume required for a site. Subsection 2.1.11 illustrates the recommended average outflow method for volume calculation.
- *Hydrograph Generation:* The SCS TR-55 hydrograph methods provided in Section 2.1.5 can be used to compute the runoff hydrograph for the 1-year, 24-hour storm.
- *Rainfall Depths:* The rainfall depth of the 1-year, 24-hour storm will vary depending on location and can be determined from the rainfall tables included in Appendix A for various locations across North Central Texas.
- *Multiple Drainage Areas*: When a development project contains or is divided into multiple outfalls, SP_v should be calculated and addressed separately for each outfall.
- *Off-site Drainage Areas:* A structural storm water control located “on-line” will need to safely bypass any off-site flows. Maintenance agreements may be required.
- *Routing/Storage Requirements:* The required storage volume for the SP_v must lie above the permanent pool elevation in storm water ponds. Wet ponds and wetlands will have permanent pools. The portion of the WQ_v above the permanent pool may be included when routing the SP_v.
- Hydraulic control structures appropriate for each storage requirement may be needed.
- *Control Orifices*: Orifice diameters for SP_v control of less than 3 inches are not recommended without adequate clogging protection (see Section 4.6). Clogging protection must be provided on all orifices.

1.2.5 Flood Control

Flood control analyses are based on the following three (3) storm events. The storm frequencies for each event shall be established in the Local Criteria section.

- *"Streambank Protection":* Either the 1- or 2-year, 24-hour storm event
- *"Conveyance"*: Either the 5-, 10-, or 25-year, 24-hour storm event
- "100-year" the 100-year, 24-hour storm event

The intent of the flood control criteria is to provide for public safety; minimize on-site and downstream flood impacts from the "Streambank Protection", "Conveyance", and "100-year" storm events; maintain the boundaries of the mapped 100-year floodplain; and protect the physical integrity of the on-site storm water controls and the downstream storm water and flood control facilities.

Flood control must be provided for on-site conveyance, as well as downstream outfalls as described in the following sections.

1.2.5.1 On-Site Conveyance

The "Conveyance" storm event is used to design standard levels of flood protection for streets, sidewalks, structures, and properties within the development. This is typically handled by a combination of conveyance systems including street and roadway gutters, inlets and drains, storm drain pipe systems, culverts, and open channels. Other storm water controls may affect the design of these systems.

The design storms used to size the various on-site conveyance systems will vary depending upon their location and function. For example, open channels, culverts, and street rights-of way are generally designed for larger events (25- to 100-year storm), whereas inlets and storm drain pipes are designed for smaller events (5- to 25-year storm). The requirements of the local jurisdiction should be obtained and utilized as shown in the Local Criteria section of this manual.

It is recommended that once the initial set of controls are selected in the iSWM Site Plan design, the full build-out 100-year, 24-hour storm be routed through the on-site conveyance system and storm water controls to determine the effects on the systems, adjacent property, and downstream areas. Even though the conveyance systems may be designed for smaller storm events, overall, the site should be designed appropriately to safely pass the resulting flows from the full build-out 100-year storm event with no flood waters entering habitable structures.

On-site flood control has many considerations for the safeguarding of people and property. On residential streets, for the "Conveyance" storm event, the safe passage of vehicular traffic is an important concern. For the 100-year storm events, traffic may be limited in order to utilize all or portions of the right-of-way for storm water conveyance in order to protect properties. As such, the effective management of storm water throughout the development for the full range of storm events is needed.

1.2.5.2 Downstream Flood Control

The downstream assessment is the first step in the process to determine if a specific development will have a flooding impact on downstream properties, structures, bridges, roadways, or other facilities. This assessment should be conducted downstream of a development to the point where the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system.. Hydrologic and hydraulic evaluations must be conducted to determine if there are areas of concerns, i.e. an increase of the Base Flood Elevations. The local jurisdiction should be consulted to obtain records and maps related to the National Flood Insurance Program and the availability of Flood Insurance Studies and Flood Insurance Rate Maps (FIRMs) which will be helpful in this assessment.

The downstream flood control criterion is based on an analysis of the "Streambank Protection" and "Conveyance" storm events, as well as the "100-year", defined as the 100-year, 24-hour storm event (denoted Q_{p100}). The local jurisdiction should quantify the frequency of the "Streambank Protection" and "Conveyance" storm events, as well as other events that may be required based on local policy or site-specific conditions, as identified in the Local Criteria section of this manual. If on-site or downstream modifications are required for downstream flood control, easements or right-of-entry agreements may need to be obtained in accordance with the Local Criteria.

Initially, the assessment will determine if the downstream receiving system has adequate capacity in its "full build-out" floodplain. To make this determination, Q_f, the runoff which the stream can handle without having an impact on downstream properties, structures, bridges, roadways, or other facilities, must be determined. There are four options by which a community can address downstream flood control. The local jurisdiction should specify in their Local Criteria which of these options are acceptable, as well as any other alternatives for downstream flood control. These options closely follow the four options for Streambank Protection.

Option 1: Determine Acceptable Downstream Conditions

The developer should provide all supporting calculations and/or documentation to show that the existing downstream conveyance system has capacity (Q_f) to safely pass the full build-out Q_{p100} discharge. Systems shown to be adequate are reflective of areas where attempts have been made to keep flood-susceptible development out of the "full build-out" floodplain through a combination of regulatory controls, storm water master planning, and incentives. This includes communities that have regulated floodplains for fully-developed conditions. This approach recognizes that the impacts of new development might not be completely mitigated at the extreme flood level and provides a much greater assurance that local flooding will not be a problem because people and structures are kept out of harm's way.

Option 2: Provide Adequate Downstream Conveyance Systems

If the downstream receiving system does not have adequate capacity, then the developer may choose to provide modifications to the off-site, downstream conveyance system. If this option is chosen the proposed modifications must be designed to adequately convey the full build-out storm water peak discharges for the three (3) storm events. The modifications must also extend to the point at which the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system. The developer must provide supporting calculations and/or documentation that the downstream peak discharges and water surface elevations are safely conveyed by the proposed system, without endangering downstream properties, structures, bridges, roadways, or other facilities.

Option 3: Install Storm Water Controls to Maintain Existing Downstream Conditions

If the downstream receiving system does not have adequate capacity, then the developer may also choose to provide storm water controls to reduce downstream flood impacts. These controls include on-site controls such as detention, regional controls, and, as a last resort, local flood protection such as levees, floodwalls, floodproofing, etc. Storm water master plans are a necessity to attempt to ensure public safety for the extreme storm event. The developer must provide supporting calculations and/or documentation that the controls will be designed and constructed so that there is no increase in downstream peak discharges or water surface elevations due to development.

Option 4: In lieu of a Downstream Assessment, Maintain Existing On-Site Runoff Conditions

Lastly, on-site controls may be used to maintain the pre-development peak discharges from the site. The developer must provide supporting calculations and/or documentation that the on-site controls will be designed and constructed to maintain on-site existing conditions.

It is important to note that Option 4 does not require a downstream assessment. It is a detention-based approach to addressing downstream flood control after the application of the *integrated* site design

practices. For many developments however, the results of a downstream assessment may show that significantly less flood control is required than "detaining to pre-development conditions". This method may also exacerbate downstream flooding problems due to timing of flows as discussed in Section 2.1.9. Therefore, it is strongly recommended that a downstream assessment be performed for all developments, and that Option 4 only be used when Options 1, 2, and 3 are not feasible.

The following items should be considered when providing downstream flood control.

- *Peak-Discharge and Hydrograph Generation:* Hydrograph methods provided in Section 2.1 can be used to compute the peak discharge rate and runoff for the three (3) storm events ("Streambank Protection", "Conveyance", and 100-year).
- *Rainfall Depths:* The rainfall depth of the three storm events will vary depending on location and can be determined from rainfall tables included in Appendix A for various locations across North Central Texas.
- *Off-site Drainage Areas:* Off-site drainage areas should be modeled as "full build-out" for the three storm events to ensure safe passage of future flows.
- *Downstream Assessment:* If flow is being detained on-site, downstream areas should be checked to ensure there is no peak flow or water surface increase above pre-development conditions to the point where the undetained discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system. More detail on Downstream Assessments is given in Section 2.1.9.

1.2.6 *integrated* Watershed Planning

1.2.6.1 Introduction

Storm water master planning is an important tool used to assess and prioritize both existing and potential future storm water problems and to consider alternative storm water management solutions. A storm water master plan is prepared to consider, in detail, what storm water management practices and measures are to be provided for an urban drainage area or a large development project.

Storm water master plans are most often used to address specific single functions such as drainage provision, flood mitigation, cost/benefit analysis, or risk assessment. These plans prescribe specific management alternatives and practices. Multi-objective storm water master planning broadens this traditional definition to potentially include land use planning and zoning, water quality, habitat, recreation, and aesthetic considerations. The broadest type of storm water master plan is the comprehensive watershed plan which is described in detail in this chapter.

For any storm water master plan, it is important at the outset to: (1) clearly identify and quantify the objectives and issues the plan will address; (2) recognize the constraints (technical, political, legal, financial, social, physical) that limit the possible solutions; and (3) develop a clear technical approach that will address the key issues and needs while staying within the constraints to potential solutions.

1.2.6.2 Types of Storm Water Master Planning

There are several basic types of storm water master plans that can be prepared. The Local Criteria section should specify whether and how master planning is applicable within the local jurisdiction. Below are descriptions of representative types of master plans.

Flood Assessment Master Plans

Flood assessment is the simplest form of storm water master planning where only the essential components, alignments, and functions of a drainage system are analyzed. The focus of these studies is on water quantity control and flood prevention and/or mitigation.

Frequently, a flood assessment study analyzes both existing conditions and projected future build-out conditions. The study is based upon estimates (usually modeled) of peak and total discharges for selected return period runoff events. The selected events should be based on local standards. Both the hydrology and hydraulics of the system are analyzed to determine water surface profiles and elevations. This, in turn, assists in determining probable locations where impacts can be expected to occur. Frequently, an alternatives analysis will be performed as part of the master plan to provide potential solutions to mitigating the flood impacts. This typically involves the modeling of proposed modifications or development scenarios.

Examples include examining the effects of detention on flooding and providing improved flood protection (e.g., flood proofing structures, levies, etc). A local community might develop HEC-HMS and HEC-RAS models for the hydrology and hydraulics of a watershed for the purposes of estimating the full build-out floodplain and regulating new development on this basis rather than the ever-changing "existing conditions" approach.

Flood Study Cost/Benefit Analysis Master Plans

Another type of master planning builds on a flood assessment master plan to determine acceptable risks and the associated costs. Using information developed in the flood analysis, economic and/or environmental impacts can be assessed. This initially entails establishing a relation between water surface elevation and associated damage (often referred to as stage-damage curves). Based on this relationship, an acceptable level of risk is determined, from which design discharges and associated water surface profiles and elevations are established. Acceptable levels of risk might be based upon the likelihood of loss of human life, impacts to residences, impacts to non-residential structures, or damage to utilities. This information then is used to determine the ultimate drainage infrastructure that will be needed to achieve the planning goals. Both a formal benefit-cost analyses and a more subjective "cost-effectiveness" approach could be used. Based on the design criteria, preliminary designs can be developed which in turn yield initial cost estimates for the infrastructure.

For example, a community might look at different flood protection strategies along a stream and estimate the costs and flood damage savings for each alternative in an effort to select the most appropriate solution(s) for that community.

Water Quality Master Plans

Master planning for storm water quality is becoming increasingly important, as nonpoint source loads are a critical component of watershed-wide water quality assessments. It may become necessary to estimate pollutant loads from storm water runoff to determine Total Maximum Daily Loads (TMDL's), as well as for the expansion of wastewater treatment facilities. A water quality master plan can provide the foundation from which to develop broader water quality assessments. Storm water quality studies will typically analyze water quality impacts to receiving waters (and groundwater) and develop structural and nonstructural strategies to reduce or minimize the pollutant loads. Studies usually involve the development, calibration, and verification of a water quality model. The level of model sophistication can vary from simple to complex. Often, a cost/benefit analysis will be performed as a component of the water quality study to quantify the efficacy of various strategies.

For example, a community might develop a simple spreadsheet-based loading model to perform planning level analyses of loadings of pollutants, potential removal by storm water controls, and the impacts of development strategies—or they may use a more complex continuous simulation water quality model and

supporting monitoring to develop a combination of point and non-point source loading estimates in support of a watershed assessment or TMDL.

Biological/Habitat Master Plans

Biological/habitat master planning is similar to a water quality master plan. However, rather than focusing on water chemistry, the focus is on the aquatic biological communities and supporting habitats. Biological assessments are being implemented on a more frequent basis to assess overall water body health. Biological studies provide the ability to assess both acute and long-term effects of nonpoint source impacts to a receiving water in the absence of continuous monitoring data. The resulting data can be used in the design and development of habitat improvements, stream restoration projects, riparian buffers, structural control retrofits, etc.

For example, a community may desire to improve the quality and aesthetics of a stream. Biological monitoring and habitat assessment establishes the baseline health of the stream and can be compared to a reference stream in the area. This information is assessed to determine causes of impairment (often paired with chemical monitoring) and methods to reduce impairment are investigated. The plan might then include riparian corridor planning, land use zoning changes, and planned habitat restoration.

Comprehensive Watershed Master Plans

The comprehensive watershed approach is the most general type of storm water master planning as well as the most extensive. The intent of a comprehensive watershed plan is to assess the health of the existing water resources and to make informed land use and storm water planning decisions. These decisions are based on the current and projected land use and development within the targeted watershed and its associated subwatersheds. Watershed-based water quantity and water quality goals are typically aimed at maintaining the pre-development hydrologic and water quality conditions to the extent practicable through peak discharge control, volume reduction, groundwater recharge, channel protection, and flood protection. In addition, watershed plans may also promote a wide range of additional goals including streambank and stream corridor restoration, habitat protection, protection of historical and cultural resources, and enhancement of recreational opportunities, aesthetic, and quality of life issues.

Watershed-based studies often involve a holistic approach to master planning, where hydrology, geomorphology, habitat, water quality, and biological community impacts are analyzed and solutions are developed.

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Section 1.3
integrated Site Design Practices

1.3.1 Overview

1.3.1.1 Introduction

The first step in addressing storm water management begins with the site planning and design process. Development projects can be designed to reduce their impact on watersheds when careful efforts are made to conserve natural areas, reduce impervious cover, and better integrate storm water treatment. By implementing a combination of these nonstructural approaches collectively known as *integrated* site design practices, it is possible to reduce the amount of runoff and pollutants that are generated from a site and provide for some nonstructural on-site treatment and control of runoff. The goals of *integrated* site design include:

- Managing storm water (quantity and quality) as close to the point of origin as possible and minimizing collection and conveyance
- Preventing storm water impacts rather than mitigating them
- Utilizing simple, nonstructural methods for storm water management that are lower cost and lower maintenance than structural controls
- Creating a multifunctional landscape
- Using hydrology as a framework for site design
- Reducing the peak runoff rates and volumes, therefore, reducing the size and cost of drainage infrastructure and structural storm water controls

Integrated site design for storm water management includes a number of site design techniques such as preserving natural features and resources, effectively laying out the site elements to reduce impact, reducing the amount of impervious surfaces, and utilizing natural features on the site for storm water management. The aim is to reduce the environmental impact "footprint" of the site while retaining and enhancing the owner/developer's purpose and vision for the site. Many of the *integrated* site design practices can reduce the cost of infrastructure while maintaining or even increasing the value of the property.

Reduction of adverse storm water runoff impacts through the use of *integrated* site design should be the first consideration of the design engineer. Operationally, economically, and aesthetically, the use of *integrated* site design practices offers significant benefits over treating and controlling runoff downstream. Therefore, all opportunities for using these methods should be explored and all options exhausted before considering structural storm water controls.

The reduction in runoff and pollutants using *integrated* site design can reduce the required runoff peak and volumes that need to be conveyed and controlled on a site and, therefore, the size and cost of necessary drainage infrastructure and structural storm water controls. In some cases, the use of *integrated* site design practices may eliminate the need for structural controls entirely. Hence, *integrated* site design practices can be viewed as both a water quantity and water quality management tool.

To provide an incentive for the use of the *integrated* Site Design Practices, point values may be assigned to each practice. Depending on the amount of points accumulated for a particular development, various types of credits can be granted by the local jurisdiction. Section 1.3.4 describes an example point system and credits in more detail. Furthermore, several of the site design practices described in this section provide a calculable reduction in the volume requirements for Water Quality Protection. Section 1.2.3.2 discusses these reduction opportunities and provides examples of their application.

The use of storm water *integrated* site design also has a number of other ancillary benefits including:

- Reduced construction costs
- Increased property values
- More open space for recreation
- More pedestrian friendly neighborhoods
- Protection of sensitive forests, wetlands, and habitats
- More aesthetically pleasing and naturally attractive landscape
- Easier compliance with wetland and other resource protection regulations

1.3.1.2 List of *integrated* Site Design Practices and Techniques

The *integrated* site design practices and techniques covered in this manual are grouped into four categories and are listed below:

- **Conservation of Natural Features and Resources**
  - Preserve Undisturbed Natural Areas
  - Preserve Riparian Buffers
  - Avoid Floodplains
  - Avoid Steep Slopes
  - Minimize Siting on Porous or Erodible Soils
- **Lower Impact Site Design Techniques**
  - Fit Design to the Terrain
  - Locate Development in Less Sensitive Areas
  - Reduce Limits of Clearing and Grading
  - Utilize Open Space Development
  - Consider Creative Designs
- **Reduction of Impervious Cover**
  - Reduce Roadway Lengths and Widths
  - Reduce Building Footprints
  - Reduce the Parking Footprint
  - Reduce Setbacks and Frontages
  - Use Fewer or Alternative Cul-de-Sacs
  - Create Parking Lot Storm Water "Islands"
- **Utilization of Natural Features for Storm Water Management**
  - Use Buffers and Undisturbed Areas
  - Use Natural Drainageways Instead of Storm Sewers
  - Use Vegetated Swale Instead of Curb and Gutter
  - Drain Rooftop Runoff to Pervious Areas

More detail on each site design practice is provided in the *integrated* Site Design Practice Summary Sheets in subsection 1.3.2. The Summary Sheets are after the work of the Center for Watershed Protection found in its 1998 publication **Better Site Design: A Handbook for changing Development Rules in Your Community**. These summaries provide the key benefits of each practice, examples, and details on how to apply them in site design.

1.3.1.3 Using *integrated* Site Design Practices

Site design should be done in unison with the design and layout of storm water infrastructure in attaining storm water management goals. Figure 1.3.1-1 illustrates the *integrated* site design process that utilizes the four *integrated* site design categories.

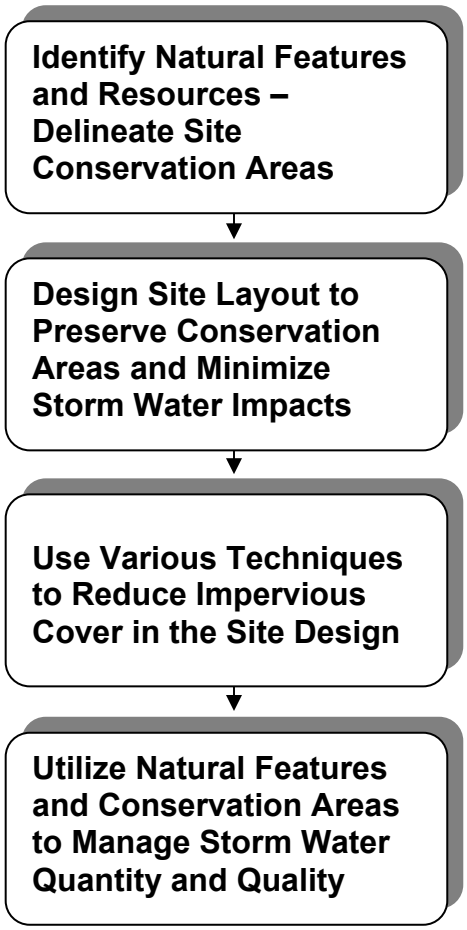


Figure 1.3.1-1 *integrated* Site Design Process

The first step in *integrated* site design involves identifying significant natural features and resources on a site such as undisturbed forest areas, stream buffers and steep slopes that should be preserved to retain some of the original hydrologic function of the site.

Next, the site layout is designed such that these conservation areas are preserved and the impact of the development is minimized. A number of techniques can then be used to reduce the overall imperviousness of the development site.

Finally, natural features and conservation areas can be utilized to serve storm water quantity and quality management purposes.

1.3.2 *integrated* Site Design Practices

1.3.2.1 Conservation of Natural Features and Resources

Conservation of natural features is integral to *integrated* site design. The first step in the *integrated* site design process is to identify and preserve the natural features and resources that can be used in the protection of water resources by reducing storm water runoff, providing runoff storage, reducing flooding, preventing soil erosion, promoting infiltration, and removing storm water pollutants. Some of the natural features that should be taken into account include:

- Areas of undisturbed vegetation
- Floodplains and riparian areas
- Ridge tops and steep slopes
- Natural drainage pathways
- Intermittent and perennial streams
- Wetlands
- Aquifers and recharge areas
- Soils
- Shallow bedrock or high water table
- Other natural features or critical areas

Some of the ways used to conserve natural features and resources described over the next several pages include the following methods:

- Preserve Undisturbed Natural Areas
- Preserve Riparian Buffers
- Avoid Floodplains
- Avoid Steep Slopes
- Minimize Siting on Porous or Erodible Soils

Delineation of natural features is typically done through a comprehensive site analysis and inventory before any site layout design is performed (see Subsection 1.1.3.4). From this site analysis, a concept plan for a site can be prepared that provides for the conservation and protection of natural features. Figure 1.3.2-1 shows an example of the delineation of natural features on a base map of a development parcel.

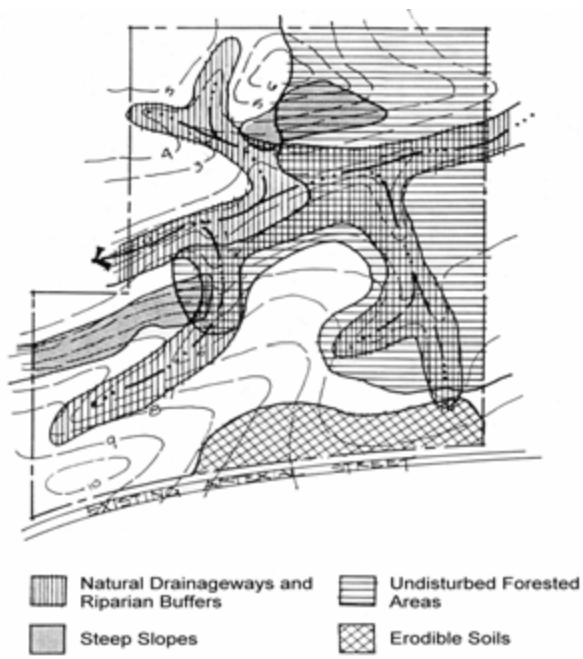


Figure 1.3.2-1 Example of Natural Feature Delineation
(Source: MPCA, 1989)

integrated Site Design Practice #1:
Preserve Undisturbed Natural Areas

Conservation of Natural Features and Resources

Description: Important natural features and areas such as undisturbed forested and vegetated areas, natural drainageways, stream corridors, wetlands and other important site features should be delineated and placed into conservation areas.

KEY BENEFITS	USING THIS PRACTICE
• Helps to preserve a portion of the site's natural predevelopment hydrology • Can be used as nonstructural storm water filtering and infiltration zones • Helps to preserve the site's natural character and aesthetic features • May increase the value of the developed property • A storm water site design credit can be taken if allowed by the local review authority	☑ Delineate natural areas before performing site layout and design ☑ Ensure that conservation areas are protected in an *undisturbed state* throughout construction and occupancy

Discussion

Preserving natural conservation areas such as undisturbed forested and vegetated areas, natural drainageways, stream corridors and wetlands on a development site helps to preserve the original hydrology of the site and aids in reducing the generation of storm water runoff and pollutants. Undisturbed vegetated areas also stabilize soils, provide for filtering and infiltration, decreases evaporation, and increases transpiration.

Natural conservation areas are typically identified through a site analysis using maps and aerial/satellite photography, or by conducting a site visit. These areas should be delineated before any site design, clearing or construction begins. When done before the concept plan phase, the planned conservation areas can be used to guide the layout of the site. Figure 1.3.2-2 shows a site map with undisturbed natural areas delineated.

Conservation areas should be incorporated into site plans and clearly marked on all construction and grading plans to ensure equipment is kept out of these areas and native vegetation is kept in an undisturbed state. The boundaries of each conservation area should be mapped by carefully determining the limit that should not be crossed by construction activity.

Once established, natural conservation areas must be protected during construction and managed after occupancy by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements. Permanent signage and fences should be required.

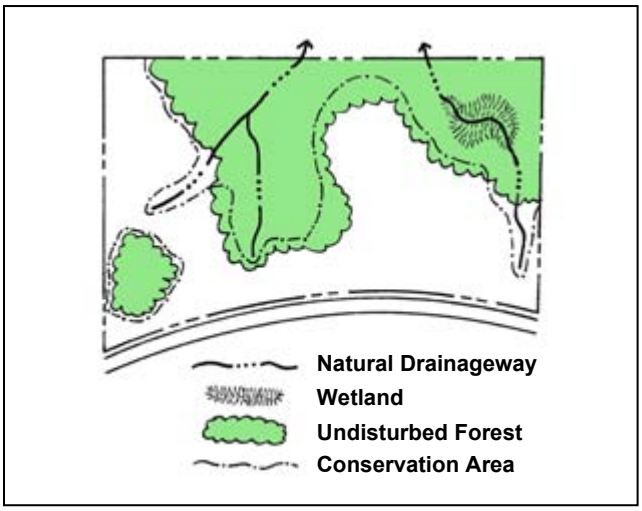


Figure 1.3.2-2 Delineation of Natural Conservation Areas

integrated Site Design Practice #2:
Preserve Riparian Buffers

Conservation of Natural Features and Resources

Description: Naturally vegetated buffers should be delineated and preserved along perennial streams, rivers, lakes, and wetlands.

KEY BENEFITS	USING THIS PRACTICE
• Can be used as nonstructural storm water filtering and infiltration zones • Keeps structures out of the floodplain and provides a right-of-way for large flood events • Helps to preserve riparian ecosystems and habitats • A storm water site design reduction credit can be taken if allowed by the local review authority	☑ Delineate and preserve naturally vegetated riparian buffers ☑ Ensure buffers and native vegetation are protected throughout construction and occupancy

Discussion

A riparian buffer is a special type of natural conservation area along a stream, wetland or shoreline where development is restricted or prohibited. The primary function of buffers is to protect and physically separate a stream, lake or wetland from future disturbance or encroachment. If properly designed, a buffer can provide storm water management functions, can act as a right-of-way during floods, and can sustain the integrity of stream ecosystems and habitats. An example of a riparian stream buffer is shown in Figure 1.3.2-3.

Forested riparian buffers should be maintained and reforestation should be encouraged where no wooded buffer exists. Proper restoration should include all layers of the forest plant community, including understory, shrubs and groundcover, not just trees. A riparian buffer can be of fixed or variable width, but should be continuous and not interrupted by impervious areas that would allow storm water to concentrate and flow into the stream without first flowing through the buffer.

Figure 1.3.2-3 Riparian Stream Buffer

Ideally, riparian buffers should be sized to include the 100-year floodplain as well as steep banks and wetlands. The buffer depth needed to perform properly will depend on the size of the stream and the surrounding conditions, but a minimum 25-foot undisturbed vegetative buffer is needed for even the smallest perennial streams and a 50-foot or larger undisturbed buffer is ideal. Even with a 25-foot undisturbed buffer, additional zones can be added to extend the total buffer to at least 75 feet from the edge of the stream. The three distinct zones within the 75-foot depth are shown in Figure 1.3.2-4. The function, vegetative target and allowable uses vary by zone as described in Table 1.3.2-1.

These recommendations are minimum standards to apply to most streams. Some streams and watershed may require additional measures to achieve protection. In some areas, specific state laws or

local ordinances already require stricter buffers than are described here. The buffer widths discussed are not intended to modify or supercede deeper or more restrictive buffer requirements already in place.

As stated above, the streamside or inner zone should consist of a minimum of 25 feet of undisturbed mature forest. In addition to runoff protection, this zone provides bank stabilization as well as shading and protection for the stream. This zone should also include wetlands and any critical habitats, and its width should be adjusted accordingly. The middle zone provides a transition between upland development and the inner zone and should consist of managed woodland that allows for infiltration and filtration of runoff. An outer zone allows more clearing and acts as a further setback for impervious surfaces. It also functions to prevent encroachment and filter runoff. In the outer zone, flow into the buffer should be transformed from concentrated flow into sheet flow to maximize ground contact with the runoff.

Development within the riparian buffer should be limited only to those structures and facilities that are absolutely necessary. Such limited development should be specifically identified in any codes or ordinances enabling the buffers. When construction activities do occur within the riparian corridor, specific mitigation measures should be required, such as larger buffers or riparian buffer improvements.

Generally, the riparian buffer should remain in its natural state. However, some maintenance is periodically necessary, such as planting to minimize concentrated flow, the removal of exotic plant species when these species are detrimental to the vegetated buffer and the removal of diseased or damaged trees.

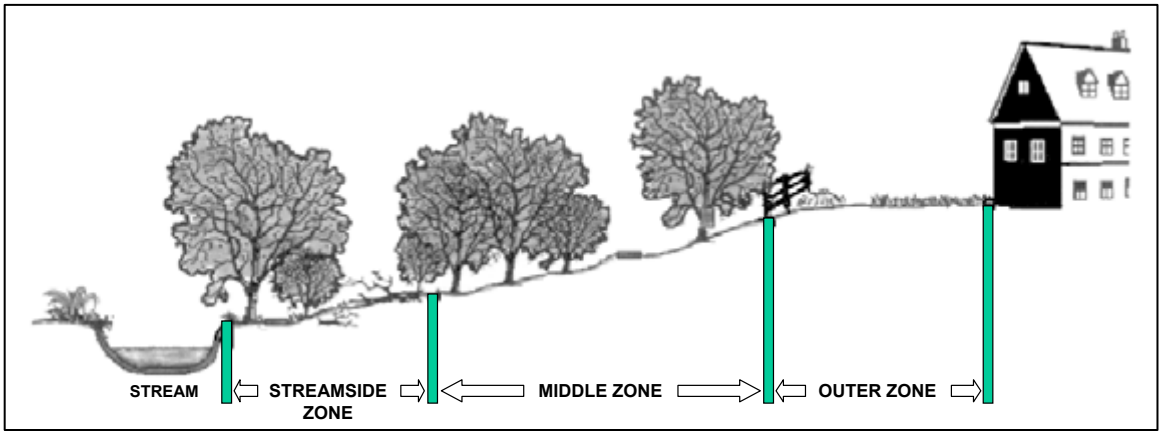


Figure 1.3.2-4 Three-Zone Stream Buffer System

Table 1.3.2-1 Riparian Buffer Management Zones

	Streamside Zone	Middle Zone	Outer Zone
Width	Minimum 25 feet plus wetlands and critical habitat	Variable depending on stream order, slope, and 100-year floodplain (min. 25 ft)	25-foot minimum setback from structures
Vegetative Target	Undisturbed mature forest. Reforest if necessary.	Managed forest, some clearing allowed.	Forest encouraged, but turf grass at a minimum
Allowable Uses	**Very Restricted** e.g., flood control, utility easements, footpaths	**Restricted** e.g., some recreational uses, some storm water controls, bike paths	**Unrestricted** e.g., residential uses including lawn, garden, most storm water controls

integrated Site Design Practice #3:
Avoid Floodplains

Conservation of Natural Features and Resources

Description: Floodplain areas should be avoided for homes and other structures to minimize risk to human life and property damage, and to allow the natural stream corridor to accommodate flood flows.

KEY BENEFITS	USING THIS PRACTICE
• Provides a natural right-of-way and temporary storage for large flood events • Keeps people and structures out of harm's way • Helps to preserve riparian ecosystems and habitats • Can be combined with riparian buffer protection to create linear greenways	☑ Obtain maps of the 100-year floodplain from the local review authority ☑ Ensure all development activities do not encroach on the designated floodplain areas

Discussion

Floodplains are the low-lying lands that border streams and rivers. When a stream reaches its capacity and overflows its channel after storm events, the floodplain provides for storage and conveyance of these excess flows. In their natural state they reduce flood velocities and peak flow rates by the passage of flows through dense vegetation. Floodplains also play an important role in reducing sedimentation by filtering runoff, and provide habitat for both aquatic and terrestrial life. Development in floodplain areas can reduce the ability of the floodplain to convey storm water, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. Most communities regulate the use of floodplain areas to minimize the risk to human life as well as to avoid flood damage to structures and property.

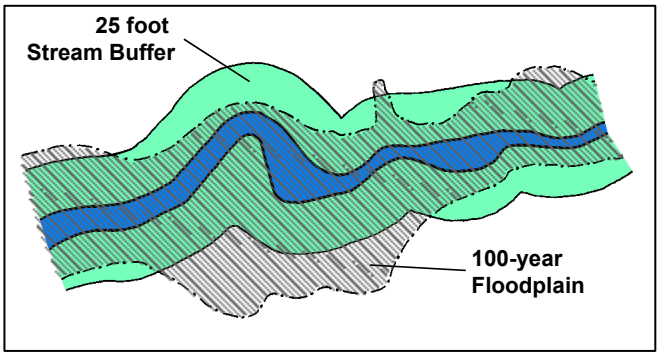


Figure 1.3.2-5 Floodplain Boundaries in Relation to a Riparian Buffer

As such, floodplain areas should be avoided on a development site. Ideally, the entire 100-year full-buildout floodplain should be avoided for clearing or building activities, and should be preserved in a natural undisturbed state where possible. Floodplain protection is complementary to riparian buffer preservation. Both of these *integrated* site design practices preserve stream corridors in a natural state and allow for the protection of vegetation and habitat. Depending on the site topography, 100-year floodplain boundaries may lie inside or outside of a preserved riparian buffer corridor, as shown in Figure 1.3.2-5.

Maps of the 100-year floodplain can typically be obtained through the local review authority. Developers and builders should also ensure their site designs comply with any other relevant local floodplain and FEMA requirements.

integrated Site Design Practice #4:
Avoid Steep Slopes

Conservation of Natural Features and Resources

Description: Steep slopes should be avoided due to the potential for soil erosion and increased sediment loading. Excessive grading and flattening of hills and ridges should be minimized.

KEY BENEFITS	USING THIS PRACTICE
• Preserving steep slopes helps to prevent soil erosion and degradation of storm water runoff quality • Steep slopes can be kept in an undisturbed natural condition to help stabilize hillsides and soils • Building on flatter areas will reduce the need for cut-and-fill and grading	☑ Avoid development on steep slope areas, especially those with a grade of 15% or greater. ☑ Minimize grading and flattening of hills and ridges

Discussion

Developing on steep slope areas has the potential to cause excessive soil erosion and increased storm water runoff during and after construction. Past studies by the SCS (now NRCS) and others have shown that soil erosion is significantly increased on slopes of 15% or greater. In addition, the nature of steep slopes means that greater areas of soil and land area are disturbed to locate facilities on them compared to flatter slopes as demonstrated in Figure 1.3.2-6.

Therefore, development on slopes with a grade of 15% or greater should be avoided if possible to limit soil loss, erosion, excessive storm water runoff, and the degradation of surface water. Excessive grading should be avoided on all slopes, as should the flattening of hills and ridges. Steep slopes should be kept in an undisturbed natural condition to help stabilize hillsides and soils. If slopes are already bare and eroding, controls to stabilize and revegetate the slopes must be considered.

On slopes greater than 25%, no development, regrading, or stripping of vegetation should be considered unless the disturbance is for roadway crossings or utility construction and it can be demonstrated that the roadway or utility improvements are absolutely necessary in the sloped area.

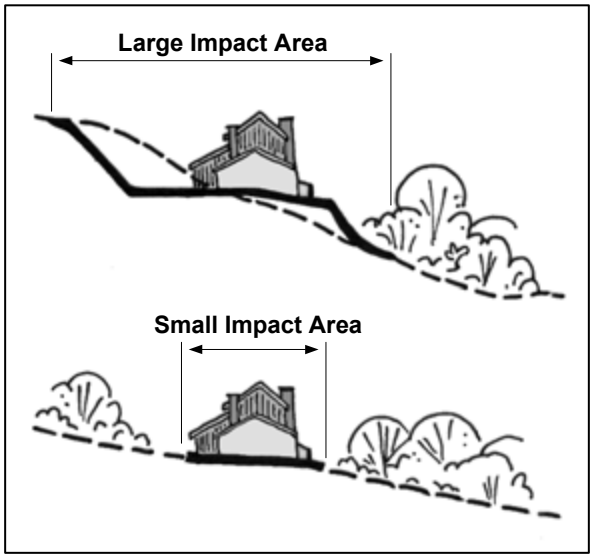


Figure 1.3.2-6 Flattening Steep Slopes for Building Sites Uses More Land Area than Building on Flatter Slopes
(Source: MPCA, 1989)

integrated Site Design Practice #5:
Minimize Siting on Permeable or Erodible Soils

Conservation of Natural Features and Resources

Description: Permeable soils such as sand and gravels provide an opportunity for groundwater recharge of storm water runoff and should be preserved as a potential storm water management option. Unstable or easily erodible soils should be avoided due to their greater erosion potential.

KEY BENEFITS	USING THIS PRACTICE
• Areas with highly permeable soils can be used as nonstructural storm water infiltration zones. A storm water site design credit can be taken if allowed by the local review authority • Avoiding highly erodible or unstable soils can prevent erosion and sedimentation problems and water quality degradation	☑ Use soil surveys to determine site soil types ☑ Leave areas of porous or highly erodible soils as undisturbed conservation areas

Discussion

Infiltration of storm water into the soil reduces both the volume and peak discharge of runoff from a given rainfall event, and also provides for water quality treatment and groundwater recharge. Soils with maximum permeabilities (hydrologic soil group A and B soils such as sands and sandy loams) allow for the most infiltration of runoff into the subsoil. Thus, areas of a site with these soils should be conserved as much as possible and these areas should ideally be incorporated into undisturbed natural or open space areas. Conversely, buildings and other impervious surfaces should be located on those portions of the site with the *least* permeable soils to the extent that soil stability, shrink-swell potential, and other soil characteristics allow.

Similarly, areas on a site with highly erodible or unstable soils should be avoided for land disturbing activities and buildings to prevent erosion and sedimentation problems as well as potential future structural problems. These areas should be left in an undisturbed and vegetated condition.

Soils on a development site should be mapped in order to preserve areas with permeable soils, and to identify those areas with unstable or erodible soils as shown in Figure 1.3.2-7. Soil surveys can provide a considerable amount of information relating to all relevant aspects of soils. Appendix B of this Manual provides permeability, shrink-swell potential and hydrologic soils group information for all North Central Texas soil series. General soil types should be delineated on concept site plans to guide site layout and the placement of buildings and impervious surfaces.

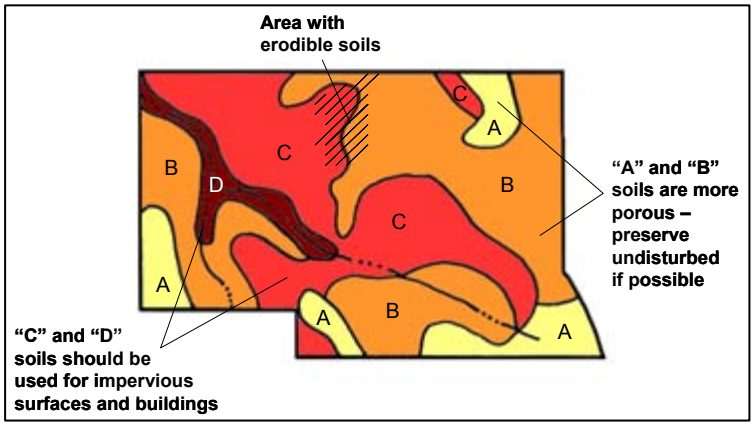


Figure 1.3.2-7 Soil Mapping Information Can Be Used to Guide Development

1.3.2.2 Lower Impact Site Design Techniques

After a site analysis has been performed and conservation areas have been delineated, there are numerous opportunities in the site design and layout phase to reduce both water quantity and quality impacts of storm water runoff. These primarily deal with the location and configuration of impervious surfaces or structures on the site and include the following practices and techniques covered over the next several pages:

- Fit the Design to the Terrain
- Locate Development in Less Sensitive Areas
- Reduce Limits of Clearing and Grading
- Utilize Open Space Development
- Consider Creative Development Design

The goal of lower impact site design techniques is to lay out the elements of the development project in such a way that the site design (i.e. placement of buildings, parking, streets and driveways, lawns, undisturbed vegetation, buffers, etc.) is optimized for effective storm water management. That is, the site design takes advantage of the site's natural features, including those placed in conservation areas, as well as any site constraints and opportunities (topography, soils, natural vegetation, floodplains, shallow bedrock, high water table, etc.) to prevent both on-site and downstream storm water impacts.

Figure 1.3.2-8 shows a development that has utilized several lower impact site design techniques in its overall layout and design.

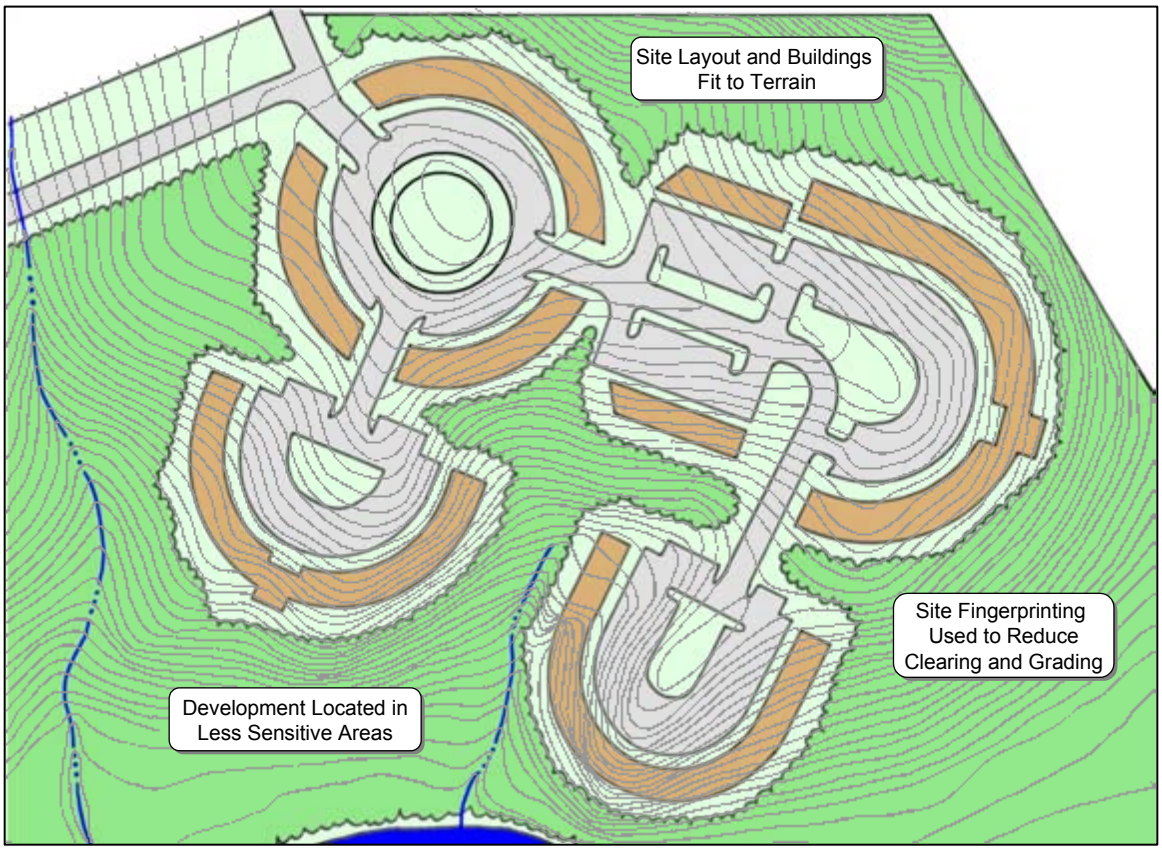


Figure 1.3.2-8 Development Design Utilizing Several Lower Impact Site Design Techniques

integrated Site Design Practice #6:
Fit Design to the Terrain

Lower Impact Site Design Techniques

Description: The layout of roadways and buildings on a site should generally conform to the landforms on a site. Natural drainageways and stream buffer areas should be preserved by designing road layouts around them. Buildings should be sited to utilize the natural grading and drainage system and avoid the unnecessary disturbance of vegetation and soils.

KEY BENEFITS	USING THIS PRACTICE
• Helps to preserve the natural hydrology and drainageways of a site • Reduces the need for grading and land disturbance • Provides a framework for site design and layout	☑ Develop roadway patterns to fit the site terrain. ☑ Locate buildings and impervious surfaces away from steep slopes, drainageways and floodplains

Discussion

All site layouts should be designed to conform with or "fit" the natural landforms and topography of a site. This helps to preserve the natural hydrology and drainageways on the site, as well as reduces the need for grading and disturbance of vegetation and soils. Figure 1.3.2-9 illustrates the placement of roads and homes in a residential development.

Roadway patterns on a site should be chosen to provide access schemes which match the terrain. In rolling or hilly terrain, streets should be designed to follow natural contours to reduce clearing and grading. Street hierarchies with local streets branching from collectors in short loops and cul-de-sacs along ridgelines help to prevent the crossing of streams and drainageways as shown in Figure 1.3.2-10. In flatter areas, a traditional grid pattern of streets or "fluid" grids which bend and may be interrupted by natural drainageways may be more appropriate (see Figure 1.3.2-11). A grid pattern may also allow for narrower streets and less imperviousness as having more than one route for emergency vehicles makes it easier to relax minimum street width requirements. In either case, buildings and impervious surfaces should be kept off of steep slopes, away from natural drainageways, and out of floodplains and other lower lying areas. In addition, the major axis of buildings should be oriented parallel to existing contours.

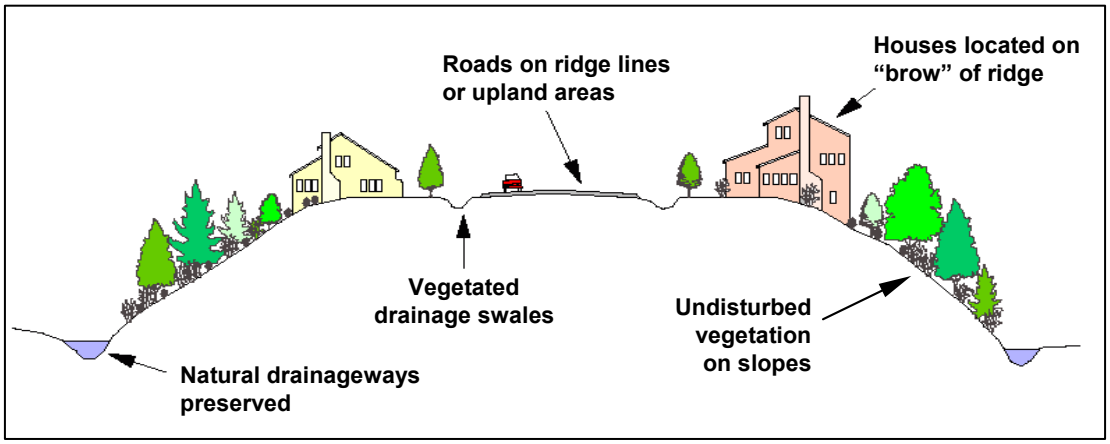


Figure 1.3.2-9 Preserving the Natural Topography of the Site
(Adapted from Sykes, 1989)

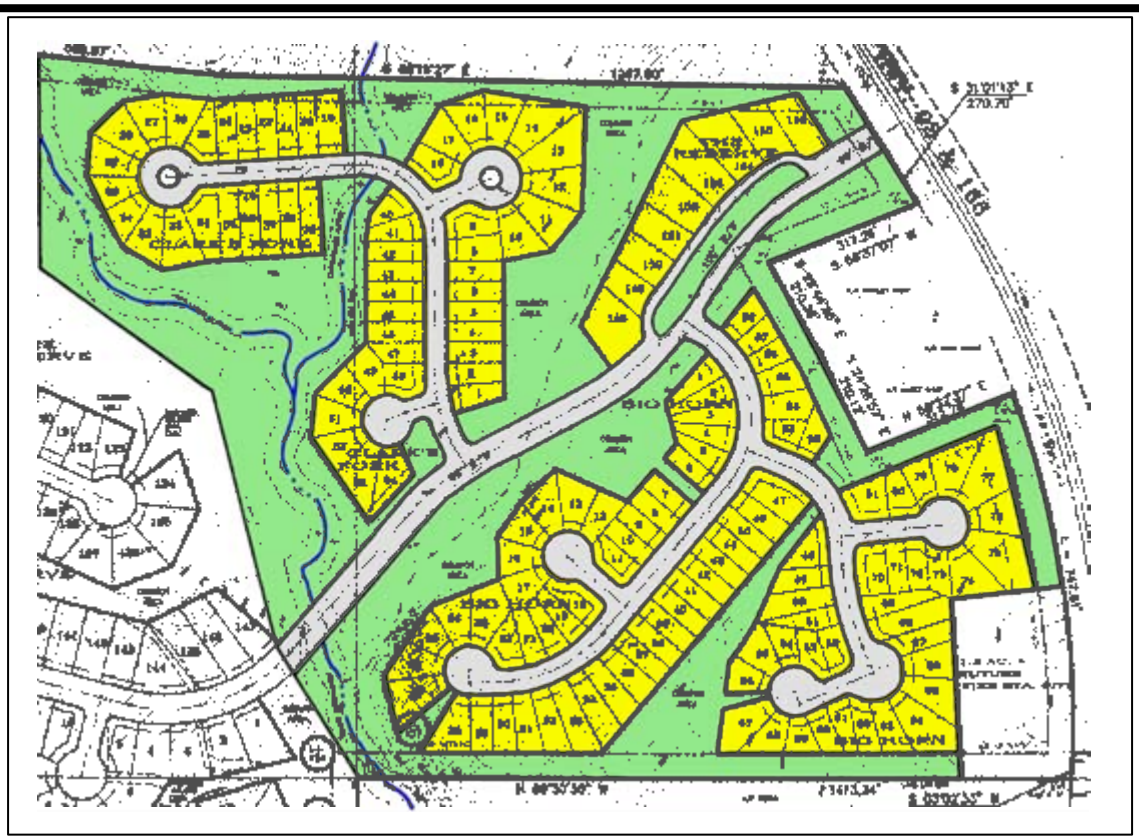


Figure 1.3.2-10 Subdivision Design for Hilly or Steep Terrain Utilizes Branching Streets from Collectors that Preserves Natural Drainageways and Stream Corridors

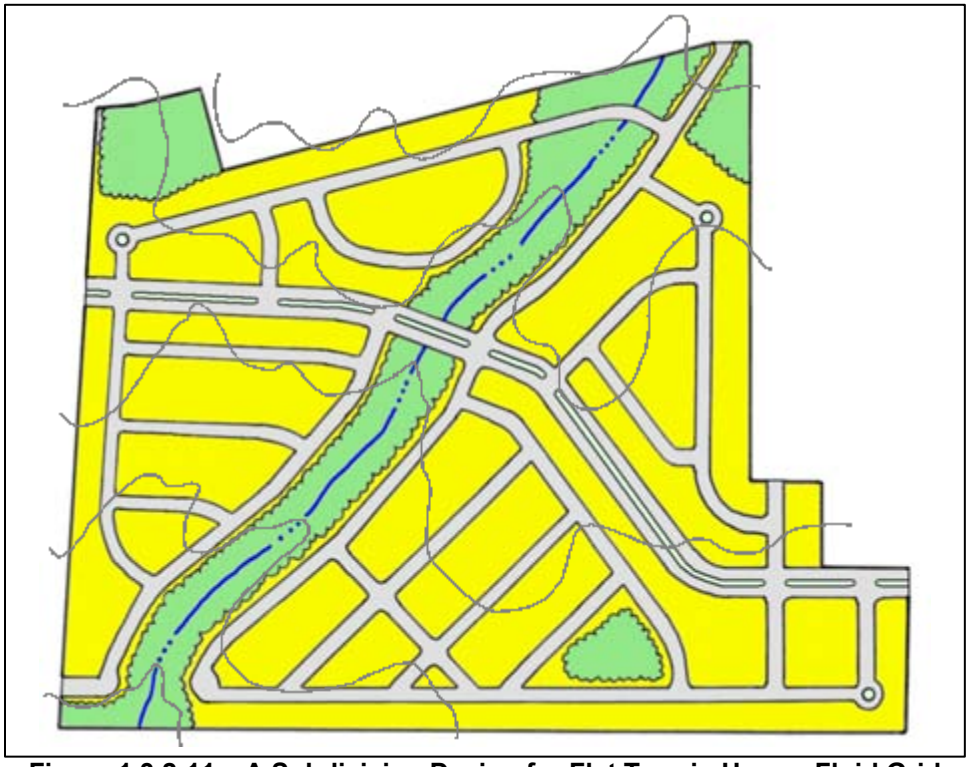


Figure 1.3.2-11 A Subdivision Design for Flat Terrain Uses a Fluid Grid Layout that is Interrupted by the Stream Corridor

integrated Site Design Practice #7:
Locate Development in Less Sensitive Areas

Lower Impact Site Design Techniques

Description: To minimize the hydrologic impacts on the existing site land cover, the area of development should be located in areas of the site that are less sensitive to disturbance or have a lower value in terms of hydrologic function.

KEY BENEFITS	USING THIS PRACTICE
• Helps to preserve the natural hydrology and drainageways of a site • Makes most efficient use of natural site features for preventing and mitigating storm water impacts • Provides a framework for site design and layout	☑ Lay out the site design to minimize the hydrologic impact of structures and impervious surfaces

Discussion

In much the same way that a development should be designed to conform to terrain of the site, a site layout should also be designed so the areas of development are placed in the locations of the site that minimize the hydrologic impact of the project. This is accomplished by steering development to areas of the site that are less sensitive to land disturbance or have a lower value in terms of hydrologic function using the following methods:

Locate buildings and impervious surfaces away from stream corridors, wetlands and natural drainageways. Use buffers to preserve and protect riparian areas and corridors.

Areas of the site with permeable soils should left in an undisturbed condition and/or used as storm water runoff infiltration zones. Buildings and impervious surfaces should be located in areas with less permeable soils.

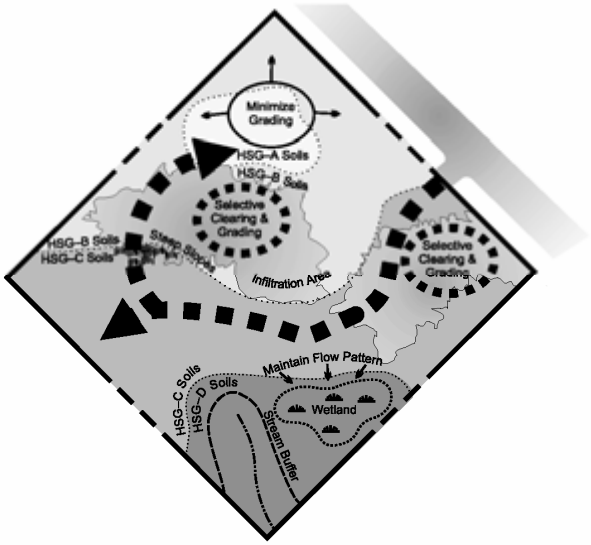


Figure 1.3.2-12 Guiding Development to Less Sensitive Areas of a Site
(Source: Prince George's County, MD, 1999)

- Avoid land disturbing activities or construction on areas with steep slopes or unstable soils.
- Minimize the clearing of areas with dense tree canopy or thick vegetation, and ideally preserve them as natural conservation areas.
- Ensure natural drainageways and flow paths are preserved, where possible. Avoid the filling or grading of natural depressions and ponding areas.

Figure 1.3.2-12 shows a development site where the natural features have been mapped in order to delineate the hydrologically sensitive areas. Through careful site planning, sensitive areas can be set aside as natural open space areas (see *integrated* Site Design Practice #9). In many cases, such areas can be used as buffer spaces between land uses on the site or between adjacent sites.

integrated Site Design Practice #8:
Reduce Limits of Clearing and Grading

Lower Impact Site Design Techniques

Description: Clearing and grading of the site should be limited to the minimum amount needed for the development and road access. Site footprinting should be used to disturb the smallest possible land area on a site.

KEY BENEFITS	USING THIS PRACTICE
• Preserves more undisturbed natural areas on a development site • Techniques can be used to help protect natural conservation areas and other site features	☑ Establish limits of disturbance for all development activities ☑ Use site footprinting to minimize clearing and land disturbance

Discussion

Minimal disturbance methods should be used to limit the amount of clearing and grading that takes place on a development site, preserving more of the undisturbed vegetation and natural hydrology of a site. These methods include:

- Establishing a limit of disturbance (LOD) based on maximum disturbance zone radii/lengths. These maximum distances should reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved.
- Using site "footprinting" which maps all of the limits of disturbance to identify the smallest possible land area on a site which requires clearing or land disturbance. Examples of site footprinting are illustrated in Figures 1.3.2-13 and 1.3.2-14.
- Fitting the site design to the terrain.
- Using special procedures and equipment which reduce land disturbance.

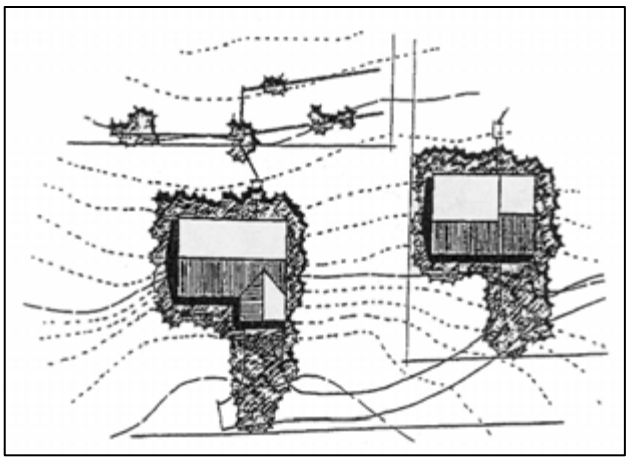


Figure 1.3.2-13 Establishing Limits of Clearing
(Source: DDNREC, 1997)

Figure 1.3.2-14 Example of Site Footprinting

integrated Site Design Practice #9:
Utilize Open Space Development

Lower Impact Site Design Techniques

Description: Open space site designs incorporate smaller lot sizes to reduce overall impervious cover while providing more undisturbed open space and protection of water resources.

KEY BENEFITS	USING THIS PRACTICE
• Preserves conservation areas on a development site • Can be used to preserve natural hydrology and drainageways • Can be used to help protect natural conservation areas and other site features • Reduces the need for grading and land disturbance • Reduces infrastructure needs and overall development costs	☑ Use a site design which concentrates development and preserves open space and natural areas of the site

Discussion

Open space development, also known as *conservation development* or *clustering*, is an *integrated* site design technique that concentrates structures and impervious surfaces in a compact area in one portion of the development site in exchange for providing open space and natural areas elsewhere on the site. Typically, smaller lots and/or nontraditional lot designs are used to cluster development and create more conservation areas on the site.

Open space developments have many benefits compared with conventional commercial developments or residential subdivisions: they can reduce impervious cover, storm water pollution, construction costs, and the need for grading and landscaping, while providing for the conservation of natural areas. Figures 1.3.2-15 and 1.3.2-16 show examples of open space developments.

Along with reduced imperviousness, open space designs provide a host of other environmental benefits lacking in most conventional designs. These developments reduce potential pressure to encroach on conservation and buffer areas because enough open space is usually reserved to accommodate these protection areas. As less land is cleared during the construction process, alteration of the natural hydrology and the potential for soil erosion are also greatly diminished. Perhaps most importantly, open space design reserves 25 to 50 percent of the development site in conservation areas, which would not otherwise be protected.

Open space developments can also be significantly less expensive to build than conventional projects. Most of the cost savings are due to reduced infrastructure cost for roads and storm water management controls and conveyances. While open space developments are frequently less expensive to build, developers also find these properties often command higher prices than those in more conventional developments. Several studies estimate that residential properties in open space developments garner premiums higher than conventional subdivisions resulting in higher selling or leasing rates.

Once established, common open space and natural conservation areas must be managed by a responsible party, typically a municipality, to maintain the areas in a natural state in perpetuity. Typically, the conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements.

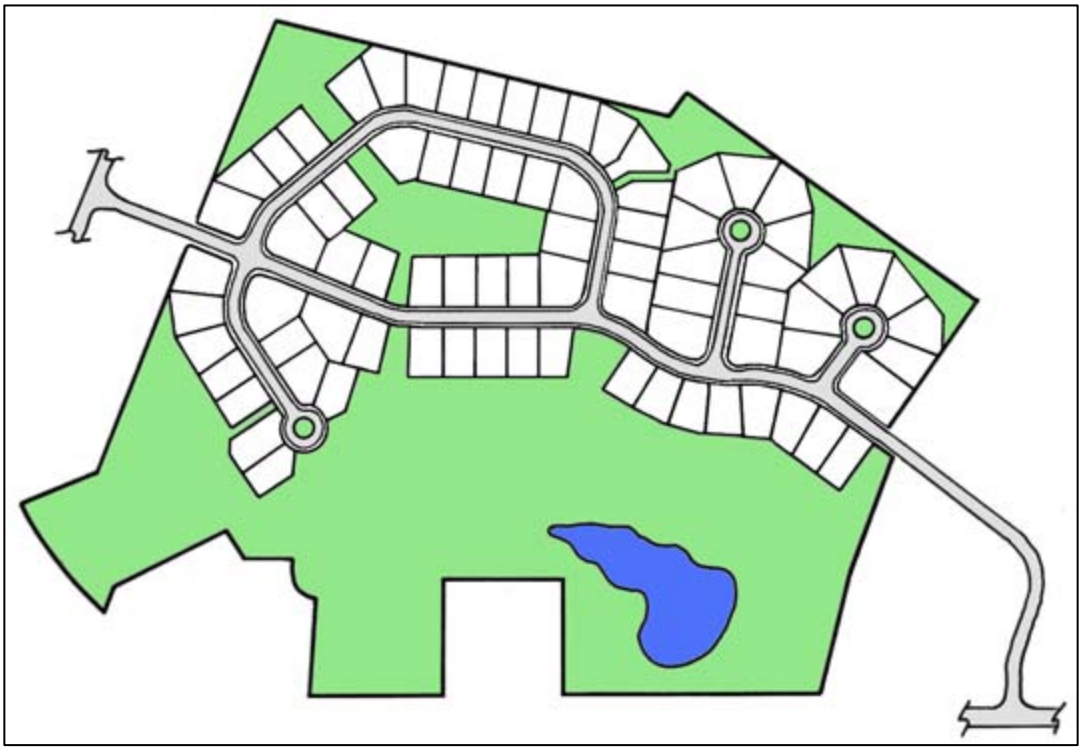


Figure 1.3.2-15 Open Space Subdivision Site Design Example

Figure 1.3.2-16 Aerial View of an Open Space Subdivision

integrated Site Design Practice #10:
Consider Creative Development Design

Lower Impact Site Design Techniques

Description: Planned Unit Developments (PUDs) allow a developer or site designer the flexibility to design a residential, commercial, industrial, or mixed-use development in a fashion that best promotes effective storm water management and the protection of environmentally sensitive areas.

KEY BENEFITS	USING THIS PRACTICE
• Allows flexibility to developers to implement creative site designs which include *integrated* site design practices • May be useful for implementing an open space development	☑ Check with your local review authority to determine if the community supports PUDs ☑ Determine the type and nature of deviations allowed and other criteria for receiving PUD approval

Discussion

A Planned Unit Development (PUD) is a type of planning approval available in some communities which provides greater design flexibility by allowing deviations from the typical development standards required by the local zoning code with additional variances or zoning hearings. The intent is to encourage better designed projects through the relaxation of some development requirements, in exchange for providing greater benefits to the community. PUDs can be used to implement many of the other *integrated* site design practices covered in this Manual and to create site designs that maximize natural nonstructural approaches to storm water management.

Examples of the types of zoning deviations which are often allowed through a PUD process include:

- Allowing uses not listed as permitted, conditional or accessory by the zoning district in which the property is located
- Modifying lot size and width requirements
- Reducing building setbacks and frontages from property lines
- Altering parking requirements
- Increasing building height limits

Many of these changes are useful in reducing the amount of impervious cover on a development site (see *integrated* Site Design Practices #11 through #16).

A developer or site designer should consult the local review authority to determine whether the community supports PUD approvals. If so, the type and nature of deviations allowed from individual development requirements should be obtained from the review authority in addition to any other criteria that must be met to obtain a PUD approval.

1.3.2.3 Reduction of Impervious Cover

The level of impervious cover, i.e. rooftops, parking lots, roadways, sidewalks and other surfaces that do not allow rainfall to infiltrate into the soil, is an essential factor to consider in *integrated* site design for storm water management. Increased impervious cover means increased storm water generation and increased pollutant loadings.

Thus by reducing the area of total impervious surface on a site, a site designer can directly reduce the volume of storm water runoff and associated pollutants that are generated. It can also reduce the size and cost of necessary infrastructure for storm water drainage, conveyance, and control and treatment. Some of the ways impervious cover can be reduced in a development include:

- Reduce Roadway Lengths and Widths
- Reduce Building Footprints
- Reduce the Parking Footprint
- Reduce Setbacks and Frontages
- Use Fewer or Alternative Cul-de-Sacs
- Create Parking Lot Storm Water Islands

Figure 1.3.2-17 shows an example of a residential subdivision that employed several of these principles to reduce the overall imperviousness of the development. The next several pages cover these methods in more detail.

Figure 1.3.2-17 Example of Reducing Impervious Cover (clockwise from upper left): (a) Cul-de-sac with Landscaped Island; (b) Narrower Residential Street; (c) "Green" Parking Lot with Landscaped Islands; and (d) Landscape Median in Roadway.

integrated Site Design Practice #11:
Reduce Roadway Lengths and Widths

Reduction of Impervious Cover

Description: Roadway lengths and widths should be minimized on a development site where possible to reduce overall imperviousness.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated • Reduces the costs associated with road construction and maintenance	☑ Consider different site and road layouts that reduce overall street length ☑ Minimize street width by using narrower street designs

Discussion

The use of alternative road layouts that reduce the total linear length of roadways can significantly reduce overall imperviousness of a development site. Site designers are encouraged to analyze different site and roadway layouts to see if they can reduce overall street length. The length of local cul-de-sacs and cross streets should be shortened to a maximum of 200 ADT (average trips per day) to minimize traffic and road noise so shorter setbacks may be employed.

In addition, residential streets and private streets within commercial and other development should be designed for the minimum required pavement width needed to support travel lanes, on-street parking, and emergency access. Figure 1.3.2-18 shows a number of different options for narrower street designs. One-way single-lane loop roads are another way to reduce the width of lower traffic streets.

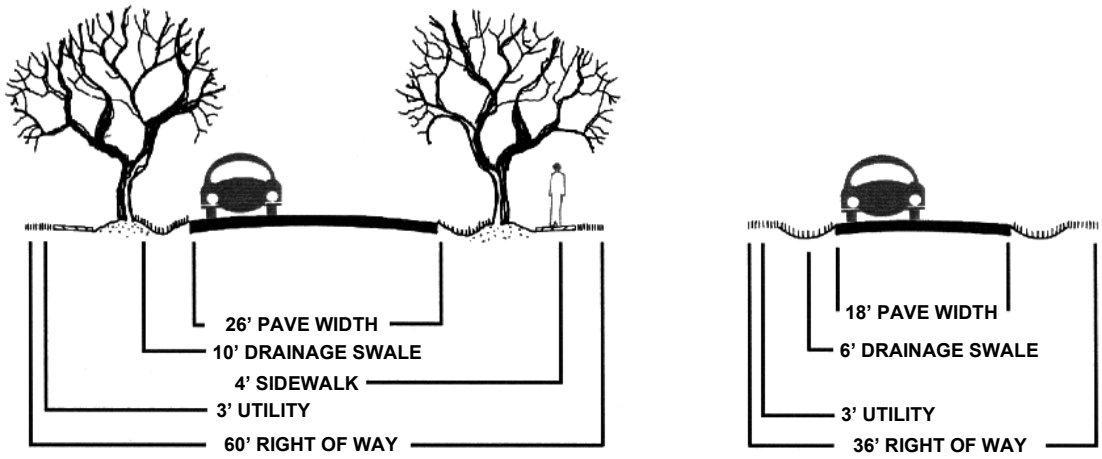


Figure 1.3.2-18 Potential Design Options for Narrower Roadway Widths
(Source: VPISU, 2000)

integrated Site Design Practice #12:
Reduce Building Footprints

Reduction of Impervious Cover

Description: The impervious footprint of commercial buildings and residences can be reduced by using alternate or taller buildings while maintaining the same floor to area ratio.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Use alternate or taller building designs to reduce the impervious footprint of buildings

Discussion

In order to reduce the imperviousness associated with the footprint and rooftops of buildings and other structures, alternative and/or vertical (taller) building designs should be considered. Consolidate functions and buildings, as required, or segment facilities to reduce the footprint of individual structures. Figure 1.3.2-19 shows the reduction in impervious footprint by using a taller building design.

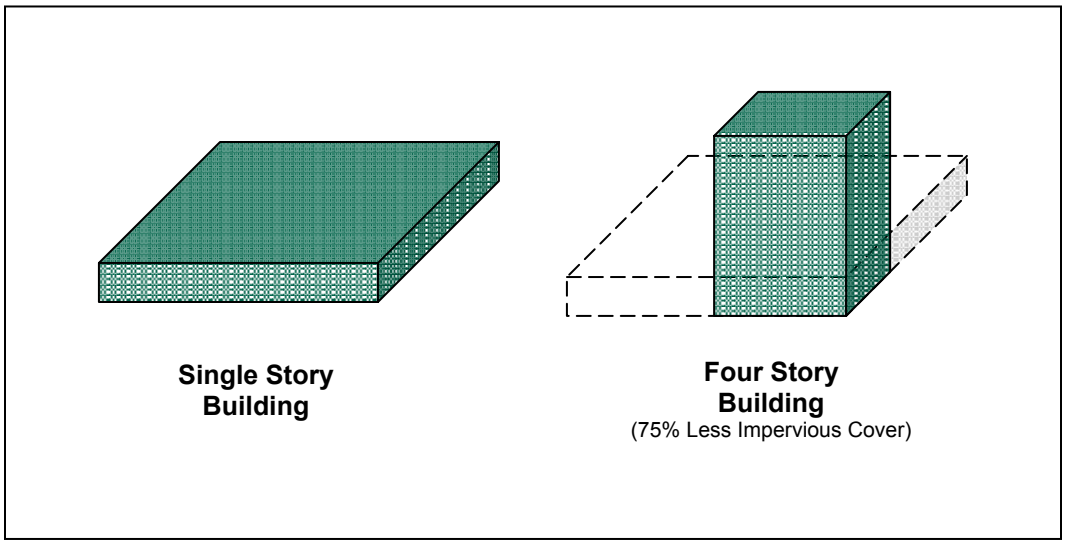


Figure 1.3.2-19 Building Up Rather Than Out Can Reduce the Amount of Impervious Cover

integrated Site Design Practice #13:
Reduce the Parking Footprint

Reduction of Impervious Cover

Description: Reduce the overall imperviousness associated with parking lots by providing compact car spaces, minimizing stall dimensions, incorporating efficient parking lanes, parking decks, and using porous paver surfaces or porous concrete in overflow parking areas where feasible and where soils allow for infiltration.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Reduce the number of parking spaces ☑ Minimize stall dimensions ☑ Consider parking structures and shared parking ☑ Use alternative porous surface for overflow areas

Discussion

Setting maximums for parking spaces, minimizing stall dimensions, using structured parking, encouraging shared parking and using alternative porous surfaces can all reduce the overall parking footprint and site imperviousness.

Sometimes parking lot designs result in far more spaces than actually required. This problem may be caused by a common practice of setting parking ratios to accommodate the highest hourly parking during the peak season. By determining average parking demand instead, a lower maximum number of parking spaces can be set to accommodate most of the demand. Table 1.3.2-2 provides examples of conventional parking requirements and compares them to average parking demand.

Table 1.3.2-2 Conventional Minimum Parking Ratios
(Source: ITE, 1987; Smith, 1984; Wells, 1994)

Land Use	Parking Requirement		Actual Average Parking Demand
	Parking Ratio	Typical Range	
Single family homes	2 spaces per dwelling unit	1.5–2.5	1.11 spaces per dwelling unit
Shopping center	5 spaces per 1000 ft^2 GFA	4.0–6.5	3.97 per 1000 ft^2 GFA
Convenience store	3.3 spaces per 1000 ft^2 GFA	2.0–10.0	--
Industrial	1 space per 1000 ft^2 GFA	0.5–2.0	1.48 per 1000 ft^2 GFA
Medical/ dental office	5.7 spaces per 1000 ft^2 GFA	4.5–10.0	4.11 per 1000 ft^2 GFA
GFA = Gross floor area of a building without storage or utility spaces.			

Another technique to reduce the parking footprint is to minimize the dimensions of the parking spaces. This can be accomplished by reducing both the length and width of the parking stall. Parking stall dimensions can be further reduced if compact spaces are provided. While the trend toward larger sport utility vehicles (SUVs) is often cited as a barrier to implementing stall minimization techniques, stall width requirements in most local parking codes are much larger than the widest SUVs.

Structured parking decks are one method to significantly reduce the overall parking footprint by minimizing surface parking. Figure 1.3.2-20 shows a parking deck used for a commercial development.

Figure 1.3.2-20 Structured Parking at an Office Park Development

Shared parking in mixed-use areas and structured parking are techniques that can further reduce the conversion of land to impervious cover. A shared parking arrangement could include usage of the same parking lot by an office space that experiences peak parking demand during the weekday with a church that experiences parking demands during the weekends and evenings.

Utilizing alternative surfaces such as porous pavers or porous concrete is an effective way to reduce the amount of runoff generated by parking lots. They can replace conventional asphalt or concrete in both new developments and redevelopment projects. Figure 1.3.2-21 is an example of porous paver used at an overflow lot. Alternative pavers can also capture and treat runoff from other site areas. However, porous pavement surfaces are generally more costly to construct and require more maintenance than conventional asphalt or concrete. For more specific information using these alternative surfaces, see the sections in Chapter 5 on (Modular Porous Paver Systems) and (Porous Concrete). These surfaces can only be used if the soils allow for adequate infiltration.

Figure 1.3.2-21 Grass Paver Surface Used for Parking

integrated Site Design Practice #14:
Reduce Setbacks and Frontages

Reduction of Impervious Cover

Description: Use smaller front and side setbacks and narrower frontages to reduce total road length and driveway lengths. This would not apply to rear access (i.e. alleys) home developments.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Reduce building and home front and side setbacks ☑ Consider narrower frontages

Discussion

Building and home setbacks should be shortened to reduce the amount of impervious cover from driveways and entry walks. A setback of 20 feet is more than sufficient to allow a car to park in a driveway without encroaching into the public right of way, and reduces driveway and walk pavement by more than 30% compared with a setback of 30 feet (see Figure 1.3.2-22).

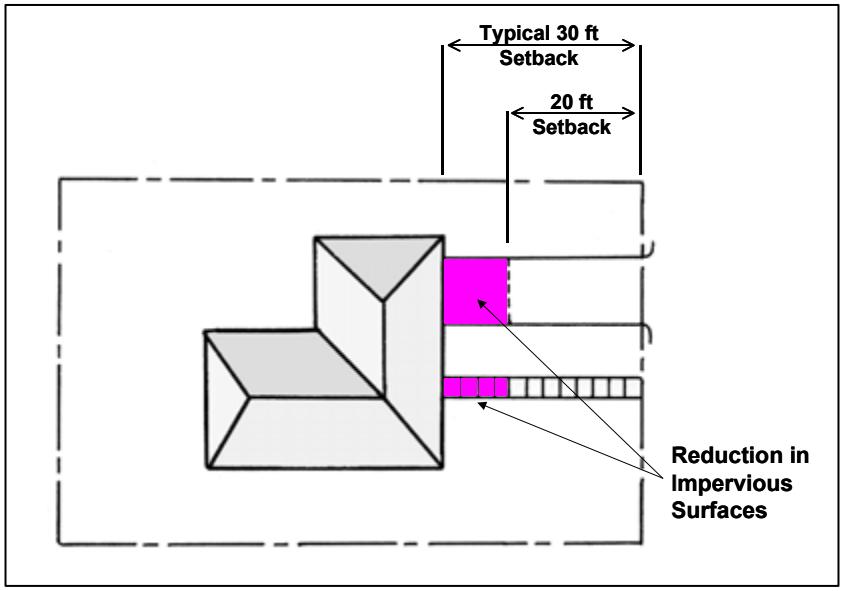


Figure 1.3.2-22 Reduced Impervious Cover by Using Smaller Setbacks
(Adapted from: MPCA, 1989)

Further, reducing side yard setbacks and using narrower frontages can reduce total street length when the same number of lots are used, especially in cluster and open space designs. Figure 1.3.2-23 shows examples of reduced front and side yard setbacks and narrow frontages.

Flexible lot shapes and setback and frontage distances allow site designers to create attractive and unique lots, which provide homeowners with enough space while allowing for the preservation of natural areas in a residential subdivision. Figure 1.3.2-24 illustrates various nontraditional lot designs.

Figure 1.3.2-23 Examples of Reduced Frontages and Side Yard Setbacks

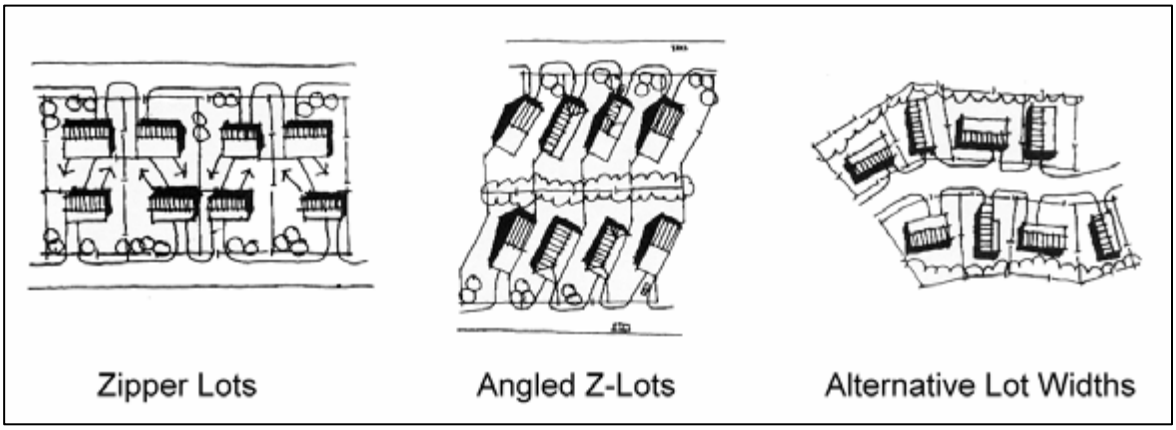


Figure 1.3.2-24 Nontraditional Lot Designs
(Source: ULI, 1992)

integrated Site Design Practice #15:
Use Fewer or Alternative Cul-de-Sacs

Reduction of Impervious Cover

Description: Minimize the number of residential street cul-de-sacs and incorporate landscaped areas to reduce their impervious cover. The radius of cul-de-sacs should be the minimum required to accommodate emergency and maintenance vehicles. Alternative turnarounds should also be considered.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Consider alternative cul-de-sac designs

Discussion

Alternative turnarounds are designs for end-of-street vehicle turnarounds that replace cul-de-sacs and reduce the amount of impervious cover created in developments. Cul-de-sacs are local access streets with a closed circular end that allows for vehicle turnarounds. Many of these cul-de-sacs can have a radius of more than 40 feet. From a storm water perspective, cul-de-sacs create a huge bulb of impervious cover, increasing the amount of runoff. For this reason, reducing the size of cul-de-sacs through the use of alternative turnarounds or eliminating them altogether can reduce the amount of impervious cover created at a site.

Numerous alternatives create less impervious cover than the traditional 40-foot cul-de-sac. These alternatives include reducing cul-de-sacs to a 30-foot radius and creating hammerheads, loop roads, and pervious islands in the cul-de-sac center (see Figure 1.3.2-25).

Sufficient turnaround area is a significant factor to consider in the design of cul-de-sacs. In particular, the types of vehicles entering into the cul-de-sac should be considered. Fire trucks, service vehicles and school buses are often cited as needing large turning radii. However, some fire trucks are designed for smaller turning radii. In addition, many newer large service vehicles are designed with a tri-axle (requiring a smaller turning radius) and many school buses usually do not enter individual cul-de-sacs.

Implementing alternative turnarounds will require addressing local regulations and marketing issues. Communities may have specific design criteria for cul-de-sacs and other alternative turnarounds that need to be modified.

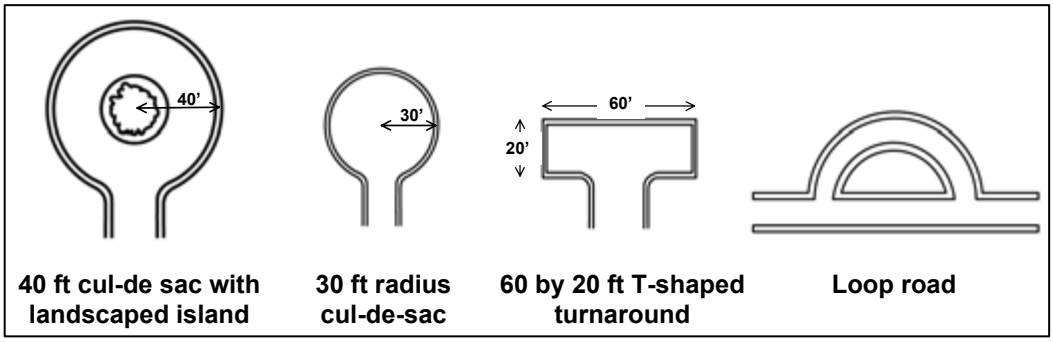


Figure 1.3.2-25 Four Turnaround Options for Residential Streets
(Source: Schueler, 1995)

integrated Site Design Practice #16:
Create Parking Lot Storm Water "Islands"

Reduction of Impervious Cover

Description: Provide storm water treatment for parking lot runoff using bioretention areas, filter strips, and/or other practices that can be integrated into required landscaping areas and traffic islands.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated • Provides an opportunity for the siting of structural control facilities • Trees in parking lots provide shading for cars and are more visually appealing	☑ Integrate porous areas such as landscaped islands, swales, filter strips and bioretention areas in a parking lot design.

Discussion

Parking lots should be designed with landscaped storm water management "islands" which reduce the overall impervious cover of the lot as well as provide for runoff treatment and control in storm water facilities.

When possible, expanses of parking should be broken up with landscaped islands which include shade trees and shrubs. Fewer large islands will sustain healthy trees better than more numerous very small islands. The most effective solutions in designing for tree roots in parking lots is to use a long planting strip at least 8 feet wide, constructed with sub-surface drainage and compaction resistant soil.

Structural control facilities such as filter strips, dry swales and bioretention areas can be incorporated into parking lot islands. Storm water is directed into these landscaped areas and temporarily detained. The runoff then flows through or filters down through the bed of the facility and is infiltrated into the subsurface or collected for discharge into a stream or another storm water facility. These facilities can be attractively integrated into landscaped areas and can be maintained by commercial landscaping firms. For detailed design specifications of filter strips, enhanced swales and bioretention areas, refer to Chapter 5.

Figure 1.3.2-26 Parking Lot Storm Water "Island"

1.3.2.4 Utilization of Natural Features for Storm Water Management

Traditional storm water drainage design tends to ignore and replace natural drainage patterns and often results in overly efficient hydraulic conveyance systems. Structural storm water controls are costly and often can require high levels of maintenance for optimal operation. Through use of natural site features and drainage systems, careful site design can reduce the need and size of structural conveyance systems and controls.

Almost all sites contain natural features that can be used to help manage and mitigate runoff from development. Features on a development site might include natural drainage patterns, depressions, permeable soils, wetlands, floodplains, and undisturbed vegetated areas that can be used to reduce runoff; provide infiltration and storm water filtering of pollutants and sediment; recycle nutrients; and maximize on-site storage of storm water. Site design should seek to utilize the natural and/or nonstructural drainage system and improve the effectiveness of natural systems rather than to ignore or replace them. These natural systems typically require low or no maintenance and will continue to function many years into the future.

Some of the methods of incorporating natural features into an overall *integrated* storm water management site plan include the following practices:

- Use Buffers and Undisturbed Areas
- Use Natural Drainageways Instead of Storm Sewers
- Use Vegetated Swales Instead of Curb and Gutter
- Drain Runoff to Pervious Areas

The following pages cover each practice in more detail.

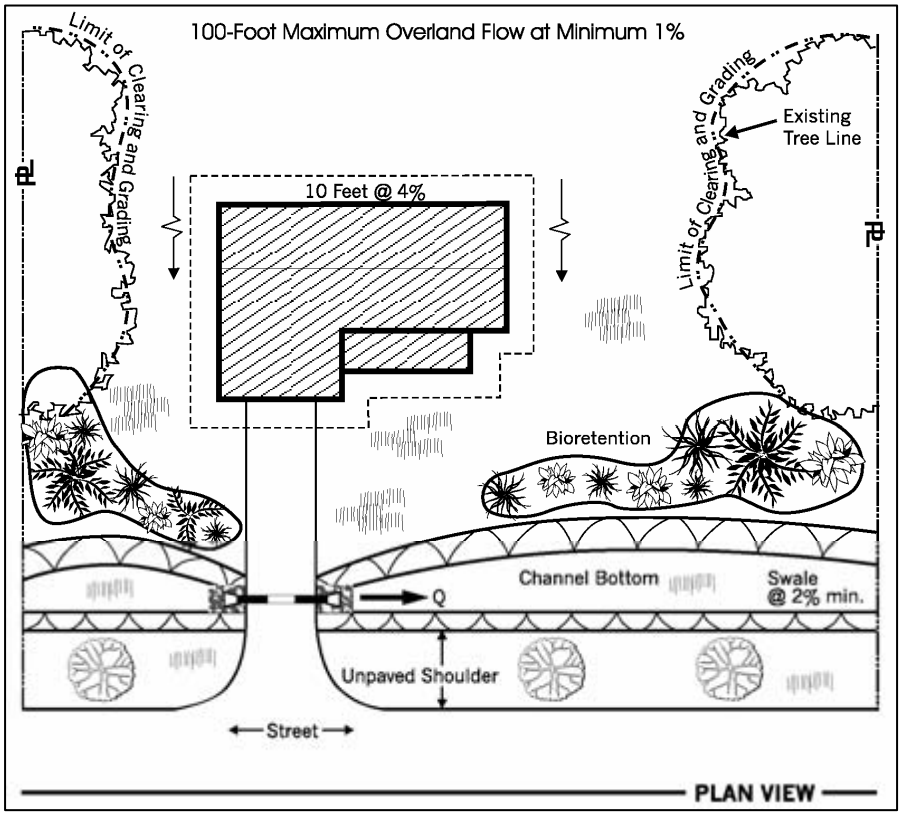


Figure 1.3.2-27 Residential Site Design Using Natural Features for Storm Water Management
(Source: Prince George's County, MD, 1999)

integrated Site Design Practice #17:
Use Buffers and Undisturbed Areas

Utilization of Natural Features for Storm Water Management

Description: Undisturbed natural areas such as forested conservation areas and stream buffers can be used to treat and control storm water runoff from other areas of the site with proper design.

KEY BENEFITS	USING THIS PRACTICE
• Riparian buffers and undisturbed vegetated areas can be used to filter and infiltrate storm water runoff • Natural depressions can provide inexpensive storage and detention of storm water flows • A storm water site design credit can be taken if allowed by the local review authority	☑ Direct runoff towards buffers and undisturbed areas using a level spreader to ensure sheet flow ☑ Utilize natural depressions for runoff storage

Discussion

Runoff can be directed towards riparian buffers and other undisturbed natural areas delineated in the initial stages of site planning to infiltrate runoff, reduce runoff velocity and remove pollutants. Natural depressions can be used to temporarily store (detain) and infiltrate water, particularly in areas with permeable (hydrologic soil group A and B) soils.

The objective in utilizing natural areas for storm water infiltration is to intercept runoff before it has become substantially concentrated and then distribute this flow evenly (as sheet flow) to the buffer or natural area. This can typically be accomplished using a level spreader, as seen in Figure 1.3.2-28. A mechanism for the bypass of higher flow events should be provided to reduce erosion or damage to a buffer or undisturbed natural area.

Carefully constructed berms can be placed around natural depressions and below undisturbed vegetated areas with pervious soils to provide for additional runoff storage and/or infiltration of flows. See the section on bioretention areas as a storm water control with a similar goal.

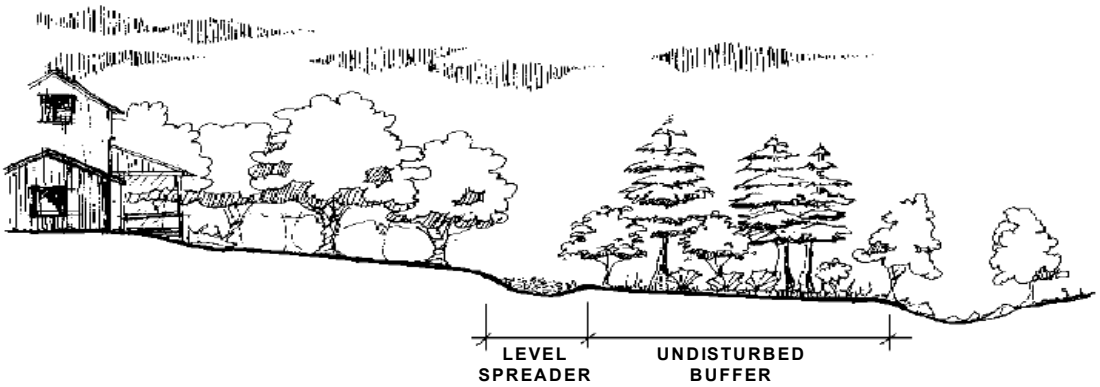


Figure 1.3.2-28 Use of a Level Spreader with a Riparian Buffer
(Adapted from NCDENR, 1998)

integrated Site Design Practice #18:
Use Natural Drainageways Instead of Storm Sewers

Utilization of Natural Features for Storm Water Management

Description: The natural drainage paths of a site can be used instead of constructing underground storm sewers or concrete open channels.

KEY BENEFITS	USING THIS PRACTICE
• Use of natural drainageways reduces the cost of constructing storm sewers or other conveyances, and may reduce the need for land disturbance and grading • Natural drainage paths are less hydraulically efficient than man-made conveyances, resulting in longer travel times and lower peak discharges • Can be combined with buffer systems to allow for storm water filtration and infiltration	☑ Preserve natural flow paths in the site design ☑ Direct runoff to natural drainageways, ensuring peak flows and velocities will not cause channel erosion

Discussion

Structural drainage systems and storm sewers are designed to be hydraulically efficient in removing storm water from a site; however, in doing so, these systems tend to increase peak runoff discharges, flow velocities and the delivery of pollutants to downstream waters. An alternative is the use of natural drainageways and vegetated swales (where slopes and soils permit) to carry storm water flows to their natural outlets, particularly for low-density development and residential subdivisions.

The use of natural open channels (see Figure 1.3.2-29) allows for more storage of storm water flows on-site, lower storm water peak flows, a reduction in erosive runoff velocities, infiltration of a portion of the runoff volume, and the capture and treatment of storm water pollutants. It is critical that natural drainageways be protected from higher post-development flows by applying downstream streambank protection methods (including the SP_v criteria) to prevent erosion and degradation.

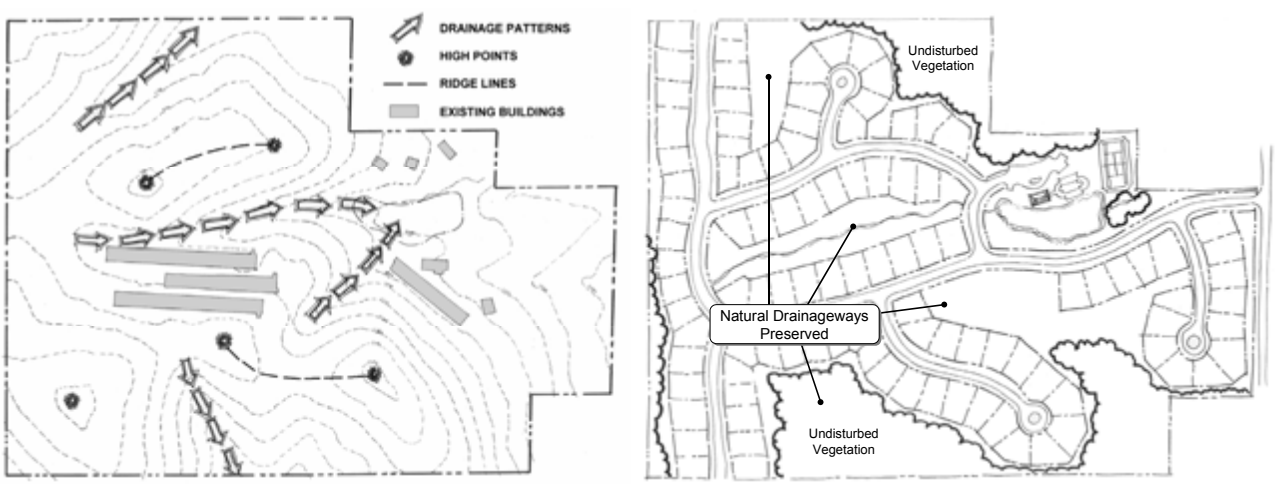


Figure 1.3.2-29 Example of a Subdivision Using Natural Drainageways for Storm Water Conveyance and Management

integrated Site Design Practice #19:
Use Vegetated Swales Instead of Curb and Gutter

Utilization of Natural Features for Storm Water Management

Description: Where density, topography, soils, slope, and safety issues permit, vegetated open channels can be used in the street right-of-way to convey and treat storm water runoff from roadways.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the cost of road and storm sewer construction • Provides for some runoff storage and infiltration, as well as treatment of storm water • A storm water site design reduction credit can be taken if allowed by the local review authority	☑ Use vegetated open channels (enhanced wet or dry swales or grass channels) in place of curb and gutter to convey and treat storm water runoff

Discussion

Curb and gutter and storm drain systems allow for quicker transport of storm water from a site to a drainageway, which results in increased peak flow and flood volumes and reduced runoff infiltration. Curb and gutter systems also do not provide treatment of storm water that is often polluted from vehicle emissions, pet waste, lawn runoff and litter.

Open vegetated channels along a roadway (see Figure 1.3.2-30) remove pollutants by allowing infiltration and filtering to occur, unlike curb and gutter systems which move water with virtually no treatment. Older roadside ditches which have not been maintained suffer from erosion, standing water, and break up of the road edge. Grass channels and enhanced dry swales are two alternatives when properly installed and maintained under the right site conditions, are excellent methods for treating storm water on-site. In addition, open vegetated channels can be less expensive to install than curb and gutter systems. Further design information and specifications for grass channels/enhanced swales can be found in Chapter 5.

Figure 1.3.2-30 Using Vegetated Swales Instead of Curb and Gutter

integrated Site Design Practice #20:
Drain Runoff to Pervious Areas

Utilization of Natural Features for Storm Water Management

Description: Where possible, direct runoff from impervious areas such as rooftops, roadways and parking lots to pervious areas, open channels or vegetated areas to provide for water quality treatment and infiltration. Avoid routing runoff directly to the structural storm water conveyance system.

KEY BENEFITS	USING THIS PRACTICE
• Sending runoff to pervious vegetated areas increases overland flow time and reduces peak flows • Vegetated areas can often filter and infiltrate storm water runoff • A storm water site design credit can be taken if allowed by the local review authority	☑ Minimize directly connected impervious areas and drain runoff as sheet flow to pervious vegetated areas

Discussion

Storm water quantity and quality benefits can be achieved by routing the runoff from impervious areas to pervious areas such as lawns, landscaping, filter strips and vegetated channels. Much like the use of undisturbed buffers and natural areas (*integrated* Site Design Practice #17), revegetated areas such as lawns and engineered filter strips and vegetated channels can act as biofilters for storm water runoff and provide for infiltration in pervious (hydrologic group A and B) soils. In this way, the runoff is "disconnected" from a hydraulically efficient structural conveyance such as a curb and gutter or storm drain system.

Some of the methods for disconnecting impervious areas include:

- Designing roof drains to flow to vegetated areas or infiltration areas
- Directing flow from paved areas such as driveways to stabilized vegetated areas
- Breaking up flow directions from large paved surfaces (see Figure 1.3.2-31)
- Carefully locating impervious areas and grading landscaped areas to achieve sheet flow runoff to the vegetated pervious areas

For maximum benefit, runoff from impervious areas to vegetated areas must occur as sheet flow and vegetation must be stabilized. See Chapter 5 for more design information and specifications on filter strips and vegetated channels.

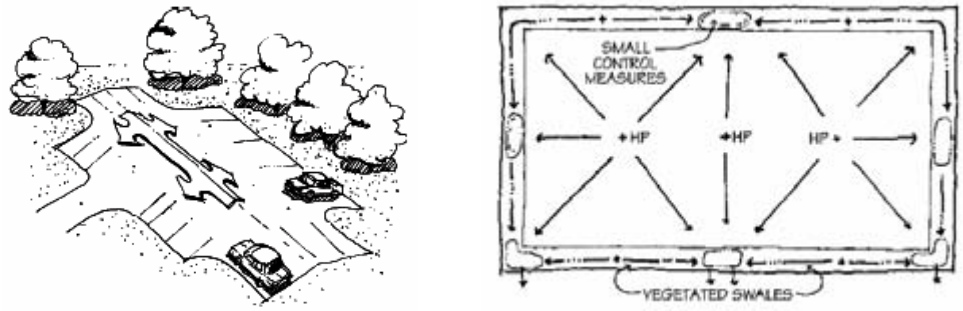


Figure 1.3.2-31 Design Paved Surfaces to Disperse Flow to Vegetated Areas
Source: NCDENR, 1998

1.3.3 *integrated* Site Design Examples

1.3.3.1 Residential Subdivision Example 1

A typical residential subdivision design on a parcel is shown in Figure 1.3.3-1 (a). The entire parcel except for the subdivision amenity area (clubhouse and tennis courts) is used for lots. The entire site is cleared and mass graded, and no attempt is made to fit the road layout to the existing topography. Because of the clearing and grading, all of the existing tree-cover, vegetation and topsoil are removed dramatically altering both the natural hydrology and drainage of the site. The wide residential streets create unnecessary impervious cover and a curb and gutter system that carries storm water flows to the storm sewer system. No provision for non-structural storm water treatment is provided on the subdivision site.

A residential subdivision employing *integrated* site design practices is presented in Figure 1.3.3-1 (b). This subdivision configuration preserves a quarter of the property as undisturbed open space and vegetation. The road layout is designed to fit the topography of the parcel, following the high points and ridgelines. The natural drainage patterns of the site are preserved and are utilized to provide natural storm water treatment and conveyance. Narrower streets reduce impervious cover and grass channels provide for treatment and conveyance of roadway and driveway runoff. Landscaped islands at the ends of cul-de-sacs also reduce impervious cover and provide storm water treatment functions. Where possible, constructing and building homes, only the building envelopes of the individual lots are cleared and graded, further preserving the natural hydrology of the site.

1.3.3.2 Residential Subdivision Example 2

Another typical residential subdivision design is shown in Figure 1.3.3-2 (a). Most of this site is cleared and mass graded, with the exception of a small riparian buffer along the large stream at the right boundary of the property. Almost no buffer was provided along the small stream that runs through the middle of the property. In fact, areas within the 100-year floodplain were cleared and filled for home sites. As is typical in many subdivision designs, this one has wide streets for on-street parking and large cul-de-sacs.

The *integrated* site design subdivision can be seen in Figure 1.3.3-2 (b). This subdivision layout was designed to conform to the natural terrain. The street pattern consists of a wider main thoroughfare, which winds through the subdivision along the ridgeline. Narrower loop roads branch off of the main road and utilize landscaped islands. Large riparian buffers are preserved along both the small and large streams. The total undisturbed conservation area is close to one-third of the site.

1.3.3.3 Commercial Development Example

Figure 1.3.3-3 (a) shows a typical commercial development containing a supermarket, drugstore, smaller shops and a restaurant on an outlot. The majority of the parcel is a concentrated parking lot area. The only pervious area is a small replanted vegetation area acting as a buffer between the shopping center and adjacent land uses. Storm water quality and quantity control are provided by a wet extended detention pond in the corner of the parcel.

An *integrated* site design commercial development can be seen in Figure 1.3.3-3 (b). Here the retail buildings are dispersed on the property, providing more of an “urban village” feel with pedestrian access between the buildings. The parking is broken up, and bioretention areas for storm water treatment are built into parking lot islands. A large bioretention area which serves as open green space is located at the main entrance to the shopping center. A larger undisturbed buffer has been preserved on the site. Because the bioretention areas and buffer provide water quality treatment, only a dry extended detention basin is needed for water quantity control.

1.3.3.4 Office Park Example

An office park with a conventional design is shown in Figure 1.3.3-4 (a). Here the site has been graded to fit the building layout and parking area. All of the vegetated areas of this site are replanted areas.

The *integrated* site design layout, presented in Figure 1.3.3-4 (b), preserves undisturbed vegetated buffers and open space areas on the site. Both the parking areas and buildings have been designed to fit the natural terrain of the site. In addition, a modular porous paver system is used for the overflow parking areas.

Figure 1.3.3-1 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).

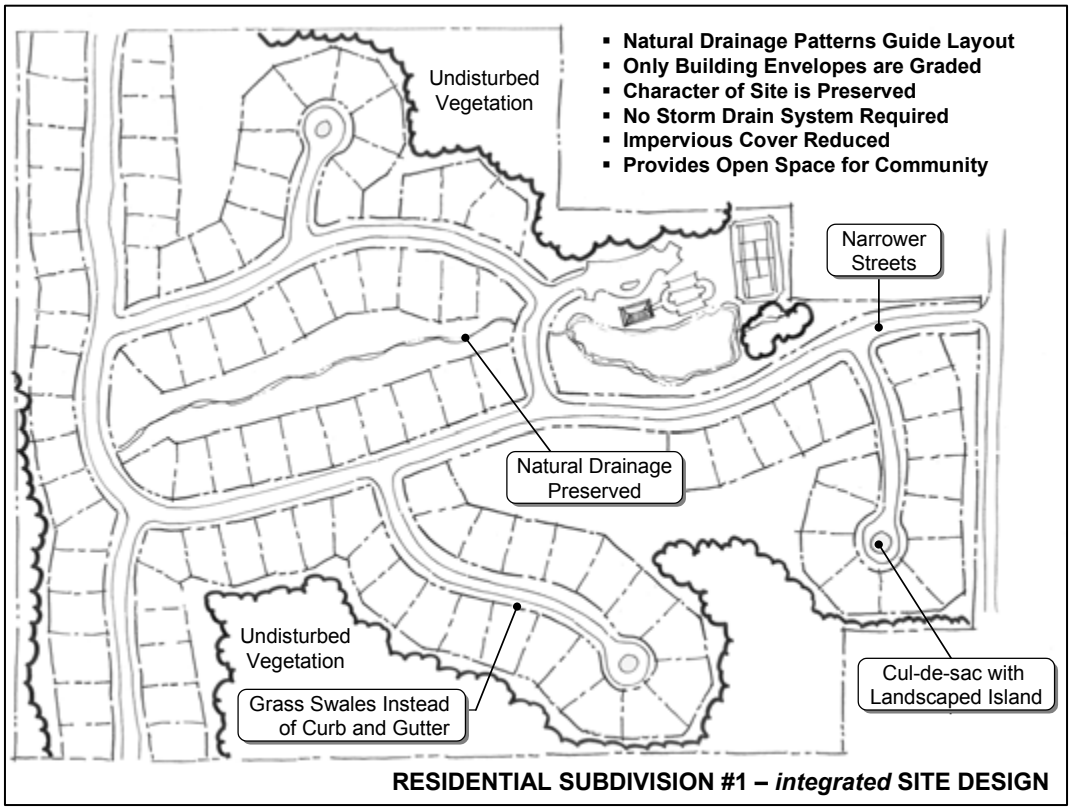
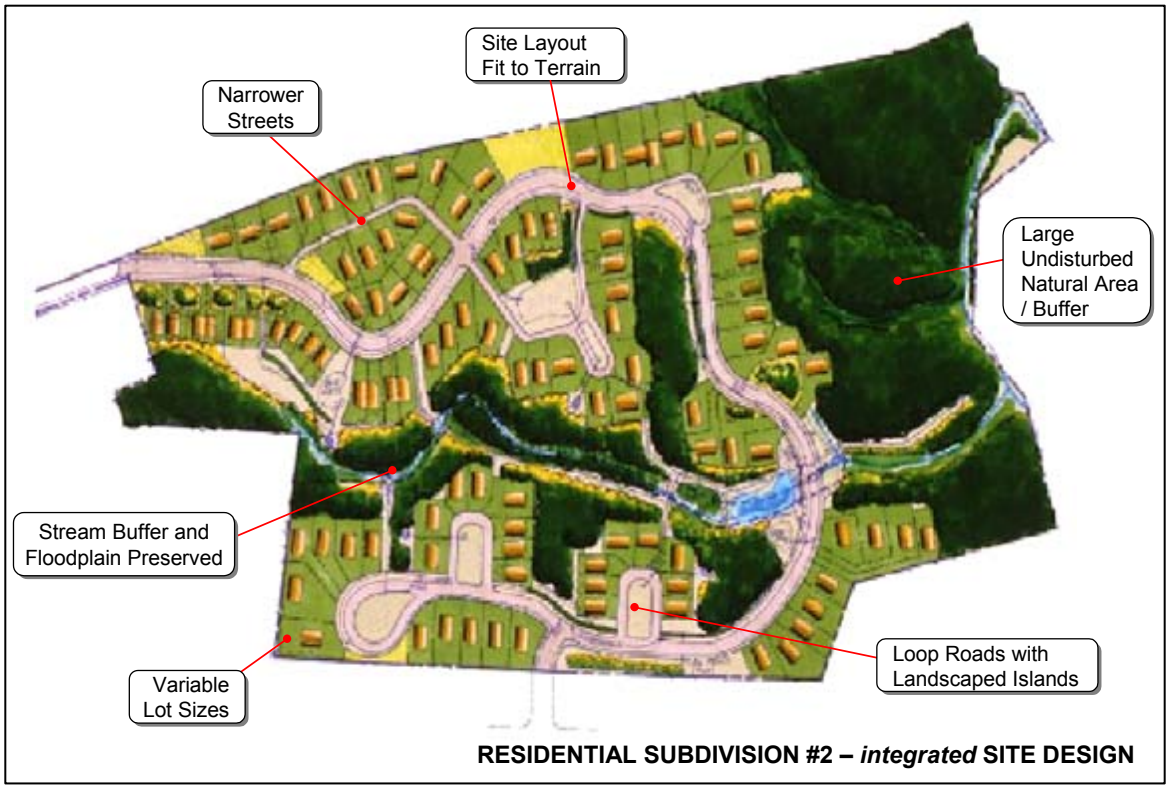


Figure 1.3.3-2 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).



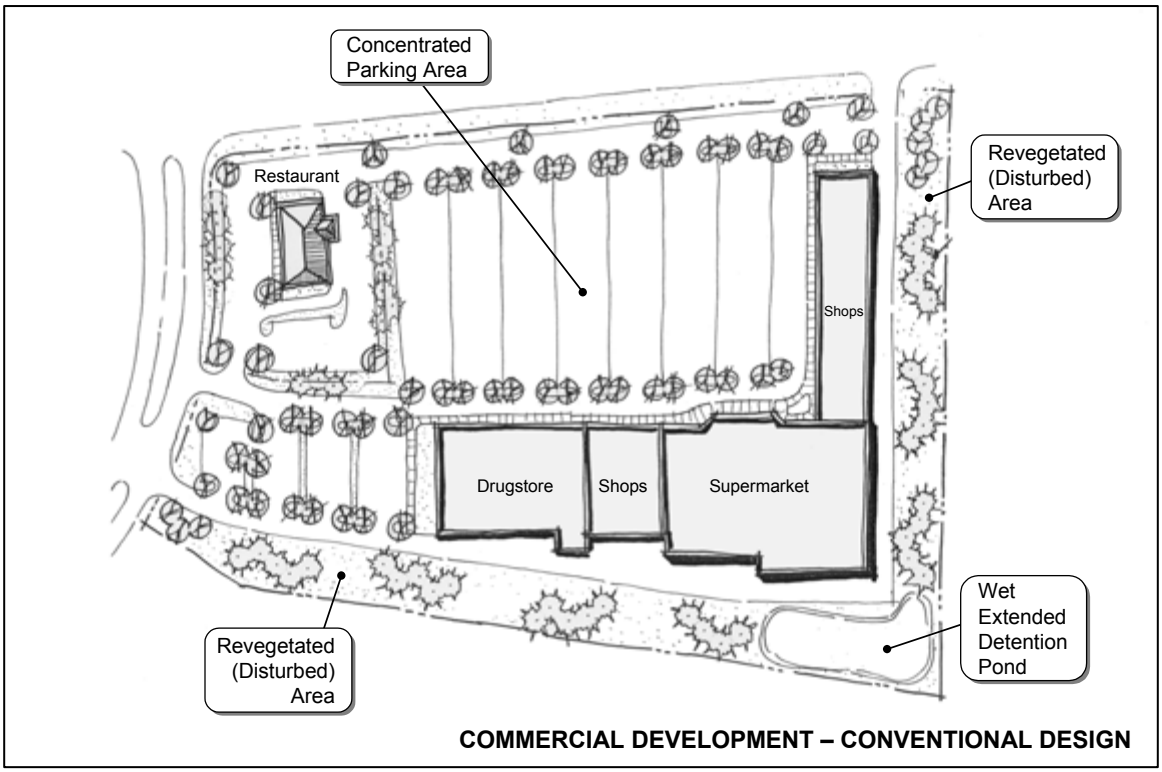
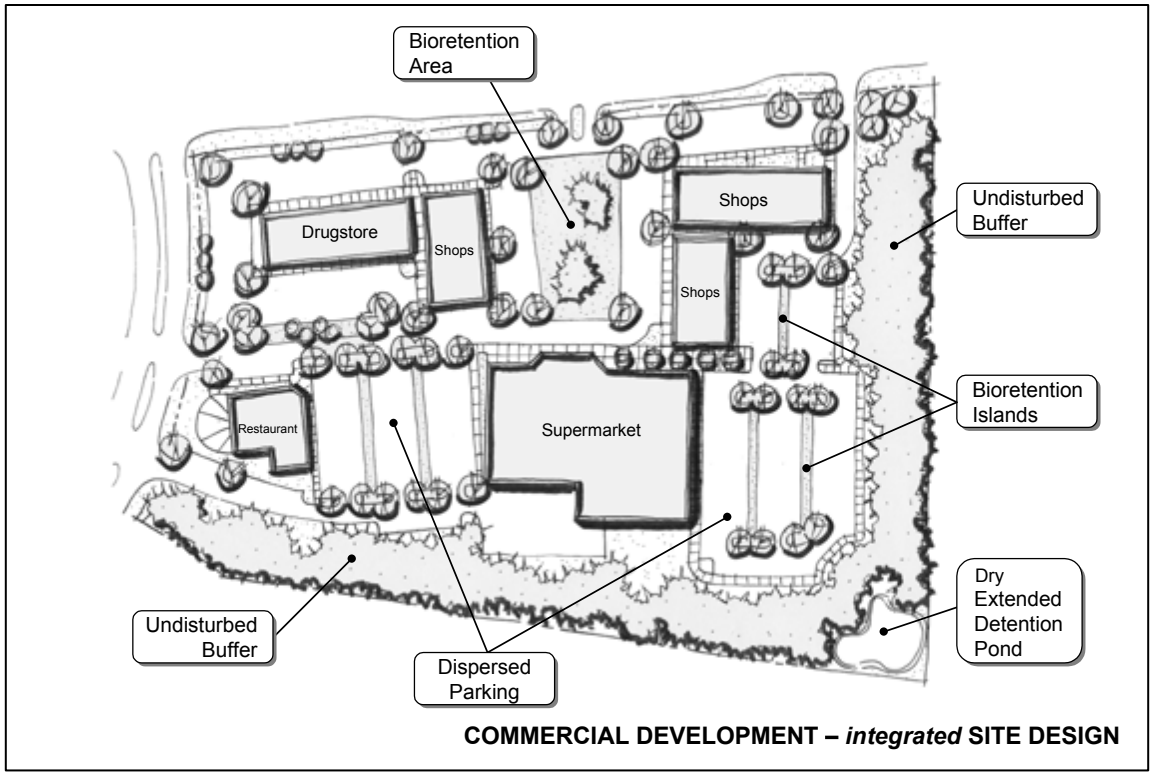


Figure 1.3.3-3 Comparison of a Traditional Commercial Development (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).



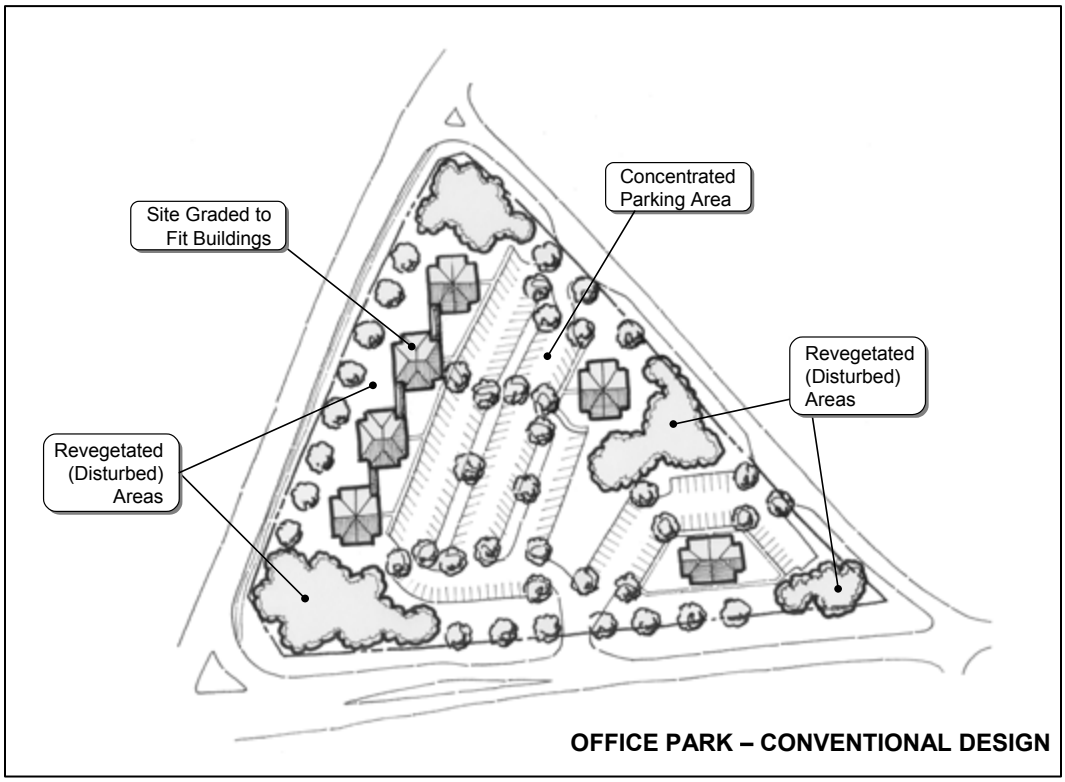
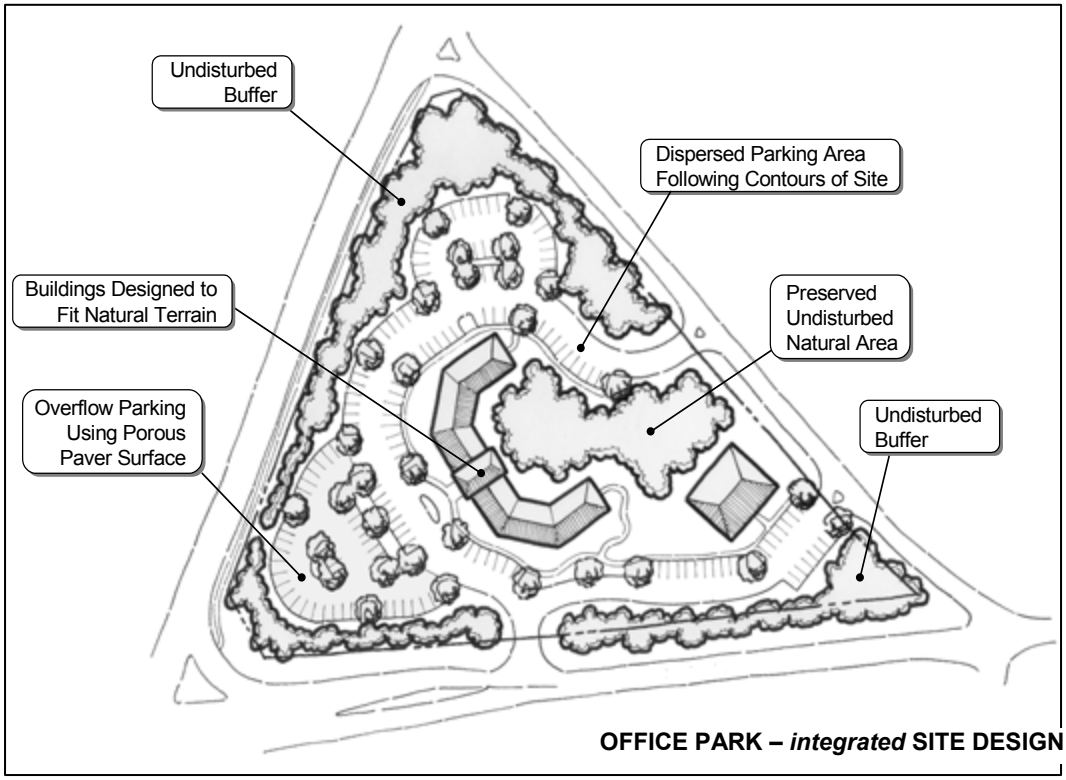


Figure 1.3.3-4 Comparison of a Traditional Office Park Design (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).



1.3.4 *integrated* Site Design Credits

1.3.4.1 Introduction

Non-structural storm water control practices are increasingly recognized as a critical feature in every site design. As such, a set of storm water "credits" has been developed to provide developers and site designers an incentive to implement *integrated* site design practices that can minimize the pollutant loads from a site.

Site designers are encouraged to utilize as many site design practices as they can on a site. Greater reductions in storm water pollutant loading can be achieved when many practices are combined (e.g., disconnecting rooftops and protecting natural conservation areas).

The type and amount of credit that is available for a development will depend on the amount of points it has accumulated, or its total "score". Multiple points can be obtained by applying one or multiple practices. Points are accumulated based on the implementation of various site design practices.

During the site planning process described in Section 1.1, there are several steps involved in site layout and design, each more clearly defining the location and function of the various components of the storm water management system. The site design practices can be integrated with this process as shown in Table 1.3.4-1.

Table 1.3.4-1 Integration of Site Design Practices with Site Development Process

Site Development Phase	Site Design Practice Activity
Feasibility Study	• Determine storm water management requirements • Perform site reconnaissance to identify potential areas for and types of credits
Site Analysis	• Identify and delineate natural feature conservation areas (natural areas and stream buffers)
Concept Plan	• Preserve natural areas and stream buffers during site layout • Reduce impervious surface area through various techniques • Identify locations for use of vegetated channels and groundwater recharge • Look for areas to disconnect impervious surfaces • Document the use of site design practices.
Preliminary and Final Plan	• Perform layout and design of credit areas – integrating them into treatment trains • Ensure *integrated* Design Approach is satisfied • Ensure appropriate documentation of site design credits according to local requirements.
Construction	• Ensure protection of key areas • Ensure correct final construction of areas needed for credits
Final Inspection	• Develop maintenance requirements and documents • Ensure long term protection and maintenance • Ensure credit areas are identified on final plan and plat if applicable

1.3.4.2 Point System

To appropriately allocate credit for using the integrated site design practices, a guideline point system has been presented. This system assigns a point value to each practice, as shown in Table 1.3.4-2. The total number of points for a given development becomes the score for the development. The score determines the type and amount of credit available to that particular development. The local jurisdiction should, as part of their Local Criteria, determine which practices (if any) they wish to give credit for. The local jurisdiction should also determine the number of points, or point weighting, assigned to each practice, and which criteria must be met in order to obtain the credit.

Table 1.3.4-2 Example Point System for *integrated* Site Design Practices

Practice	Maximum Points	Point Allocation Based on % of Practice Utilized					
		0%	<15	<30	<50	<75	>75
Conservation of Natural Features and Resources							
Preserve Undisturbed Natural Areas	5	0	1	2	3	4	5
Preserve Riparian Buffers	5	0	1	2	3	4	5
Avoid Floodplains	5	0	1	2	3	4	5
Avoid Steep Slopes	5	0	1	2	3	4	5
Minimize Siting on Porous or Erodible Soils	5	0	1	2	3	4	5
Lower Impact Site Design Techniques							
Fit Design to the Terrain	5	0	1	2	3	4	5
Locate Development in Less Sensitive Areas	5	0	1	2	3	4	5
Reduce Limits of Clearing and Grading	5	0	1	2	3	4	5
Utilize Open Space Development	5	0	1	2	3	4	5
Consider Creative Design	5	0	1	2	3	4	5
Reduction of Impervious Cover							
Reduce Roadway Lengths and Widths	5	0	1	2	3	4	5
Reduce Building Footprints	5	0	1	2	3	4	5
Reduce the Parking Footprint	5	0	1	2	3	4	5
Reduce Setbacks and Frontages	5	0	1	2	3	4	5
Use Fewer or Alternative Cul-de-Sacs	5	0	1	2	3	4	5
Create Parking Lot Storm Water "Islands"	5	0	1	2	3	4	5
Utilization of Natural Features for Storm Water Management							
Use Buffers and Undisturbed Areas	5	0	1	2	3	4	5
Use Natural Drainageways Instead of Storm Sewers	5	0	1	2	3	4	5
Use Vegetated Swale Instead of Curb and Gutter	5	0	1	2	3	4	5
Drain Rooftop Runoff to Pervious Areas	5	0	1	2	3	4	5
Total Points	100						

1.3.4.3 Example Design Credits

There are various areas of credit available for use in a storm water management program. The list below is only an example of areas of credit and should not be considered comprehensive.

- Reduction in Storm Water Utility fees
- Reduction in Park dedication requirements
- Reduction in Open space dedication requirements
- Reduction in Landscape requirements
- Decrease in plan approval process time
- Simplified planning and variance process for implementing storm water practices
- Ability to receive variance in parking or lot size requirements
- Reduction in Storm Water Quality Volume (see Section 1.2.3.3)
- Other City-Developer Agreements

Depending on the local storm water management program, some of these site design credits may be restricted, or no credits may be available. It is up to the local jurisdiction to define the types and amounts of credit they will offer to a development. Designers are encouraged to consult with the appropriate approval authority to ensure if and when a credit is applicable and to determine restrictions and guidelines on non-structural strategies.

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Section 1.4
integrated Storm Water Controls

1.4.1 Introduction

The impacts of storm water runoff from development cannot always be completely mitigated by land use and nonstructural approaches. Therefore, the use of appropriate structural storm water controls on development sites is sometimes necessary as an integrated part of the storm water management system. Storm water controls (sometimes referred to as *best management practices* or *BMPs*) are constructed storm water management facilities designed to treat storm water runoff and/or mitigate the effects of increased storm water runoff peak rate, volume, and velocity due to urbanization.

Chapter 5 recommends a number of structural storm water controls that can be used for meeting the *integrated* design approach including very specific performance and design criteria. The next several pages provide a brief overview of the range of storm water controls recommended for use in North Central Texas communities. Clearly not every control is applicable for every site or goal.

1.4.2 Recommended Storm Water Control Practices for North Central Texas Communities

Bioretention Areas

- Bioretention areas are shallow storm water basins or landscaped areas that utilize engineered soils and vegetation to capture and treat storm water runoff. Runoff may be returned to the conveyance system, or allowed to fully or partially infiltrate into the soil.

Channels

- *Enhanced Swale:* A vegetated open channel that is explicitly designed and constructed to capture and treat storm water runoff within wet or dry cells formed by check dams or other means.
- *Grass Channel:* A vegetated open channel designed to filter storm water runoff and meet velocity targets for the water quality and streambank protection design storm events.
- *Open Conveyance Channel:* Includes such conveyance systems as drainage ditches, grass channels, dry and wet enhanced swales, riprap channels, and concrete channels.

Chemical Treatment

- *Alum Treatment System:* This chemical treatment provides for the injection of liquid alum into storm water runoff on a flow-weighted basis during rain events as it enters a settling basin. The alum precipitate or 'floc' that is formed during coagulation combines with nutrients, suspended solids, and heavy metals and settles in the settling basin.

Conveyance Components

- *Culverts:* Typically, short, closed (covered) conduits that convey storm water runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water, but it may

also be used to restrict flow and reduce downstream peak flows.

- *Energy Dissipators:* Energy dissipaters are engineered devices such as riprap or concrete baffles placed at the outlet of a storm water conveyance for the purpose of reducing the velocity, energy, and turbulence of the discharged flow.
- *Inlets/Street Gutters:* Drainage elements that remove runoff from sidewalks, streets, and sumps for public safety purposes and function to input storm water to the storm drain pipe systems.
- *Pipe Systems:* A branching system of closed conduits that accumulate storm water runoff and convey it to an open channel, natural stream, or storage facility.

Detention

- **Dry Detention:* Dry detention basins are surface storage basins or facilities typically designed to provide water quantity control through detention or extended detention of storm water runoff.
- **Extended Dry Detention Basins:* Extended dry detention basins are surface storage basins or facilities that can be designed to provide water quality and quantity control through extended detention of storm water runoff.
- *Multi-Purpose Detention Areas:* Multi-purpose detention areas are facilities designed primarily for another purpose, such as parking lots and rooftops, that can provide water quantity control through detention of storm water runoff.
- *Underground Detention:* Underground detention storage is provided by underground tanks or vaults designed to provide water quantity control through detention and/or extended detention of storm water runoff.

Filtration

- *Filter Strip:* Filter strips are uniformly graded and densely vegetated sections of land engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration.
- *Organic Filter:* Organic filters are design variant of the surface sand filter using organic materials such as peat or compost in the filter media.
- *Planter Boxes:* Planter boxes are used on impervious surfaces to collect and detain/infiltrate rainfall and runoff. They usually contain growing plants.
- *Surface Sand Filter / Perimeter Sand Filter:* Sand filters are multi-chamber structures designed to treat storm water runoff through filtration, using a sand bed as the primary filter media. Filtered runoff may be returned to the conveyance system, or allowed to fully or partially infiltrate into the soil.
- *Underground Sand Filter:* The underground sand filter is a design variant of the surface sand filter located in an underground vault designed for high density land use where there is not enough space for a surface sand filter or other storm water controls.

Hydrodynamic Devices

- *Gravity (Oil-Grit) Separator:* The gravity (oil-grit) separator is a hydrodynamic separation device designed to remove settleable solids, oil, grease, debris, and floatables from storm water runoff through gravitational settling and trapping of pollutants.

Infiltration

- *Downspout Drywells:* Downspout Drywells are essentially perforated manholes, but they can be manufactured in various sizes. Located underground, they allow storm water infiltration even in highly urbanized areas. They should be used in conjunction with some type of pretreatment devices where there are minimal risks of groundwater contamination.
- *Infiltration Trench:* Infiltration trenches are excavated trenches filled with stone aggregate used to

capture and allow infiltration of storm water runoff into the surrounding soils from the bottom and sides of the trench.

- *Soakage Trench:* Soakage trenches are a variation of infiltration trenches. Soakage trenches drain through a perforated pipe buried in gravel. They are used in highly impervious areas where conditions do not allow surface infiltration and where pollutant concentrations in runoff are minimal (i.e. non-industrial rooftops). They may be used in conjunction with other storm water devices, such as draining downspouts or planter boxes.

*Ponds

There are two storm water storage functions: detention and retention. Detention ponds are designed to store water and release it over time to empty the basin. Retention basins have a permanent pool (or micropool) of water. Some basins are designed to include both detention and retention. Runoff from each rain event is detained and treated in the pool. Pond design variants include:

- Micropool Extended Detention Pond
- Multiple Pond Systems
- Wet Extended Detention Pond
- Wet Pond

Porous Surfaces

- *Green Roofs:* A green roof uses a small amount of substrate over an impermeable membrane to support a covering of plants. The green roof slows down runoff from the otherwise impervious roof surface as well as moderating rooftop temperatures. With the right plants, a green roof will also provide aesthetic or habitat benefits. Green roofs have been used in Europe for decades.
- *Modular Porous Paver Systems:* Modular porous paver systems are pavement surfaces composed of structural units with void areas that are filled with pervious materals such as sand or grass turf. Porous pavers are installed over a gravel base course to provide storage as runoff infiltrates through the porous paver system into underlying permeable soils.
- *Porous Concrete:* Porous concrete is the term for a mixture of coarse aggregate, Portland cement, and water that allows for rapid infiltration of water and overlays a stone aggregate reservoir. The reservoir provides temporary storage as runoff infiltrates into underlying permeable soils and/or out through an underdrain system.

Proprietary Structural Controls

There are numerous manufactured structural control systems available from commercial vendors designed to treat storm water runoff and/or provide water quantity control.

Re-Use

- *Rain Harvesting (Tanks/Barrels):* A rain harvesting system is a container or system designed to capture and store rainwater discharged from a roof. The rain harvesting system consists of a storage container, a downspout diversion, a sealed lid, and an overflow system. Typical rain harvesting systems hold between 50 and 500 gallons of water, and may work in series to provide larger volumes of storage. For larger roof tops an underground storage tank and pump system can provide water for irrigation purposes on site.

Wetlands

- **Storm Water Wetlands:* Storm water wetlands are constructed wetland systems used for storm water management. Storm water wetlands consist of a combination of shallow marsh areas, open water areas, and semi-wet areas above the permanent water surface. Wetland design variants include:

- Extended Detention Shallow Wetland
- Pocket Wetland
- Pond/Wetland Systems
- Shallow Wetland

- *Submerged Gravel Wetlands:* Submerged gravel wetlands are also known as subsurface flow wetlands and consist of one or more cells filled with crushed rock designed to support wetland plants. Storm water runoff flows subsurface through the root zone of the constructed wetland where pollutant removal takes place.

* *Consideration must be given in the design of storm water ponds, wetlands, and detention basins to minimize potential mosquito breeding areas. This can be accomplished in a variety of ways including aquatic and chemical techniques which should be utilized as appropriate for the situation.*

1.4.3 Suitability of Storm Water Controls to Meet Storm Water Management Goals

Table 1.4.3-1 summarizes the storm water management suitability of the various storm water controls in addressing the *integrated* Design Approach. Given that some storm water controls cannot alone meet all of the design requirements, typically two or more controls are used in series to form what is known as a storm water "treatment train." Chapter 5 provides guidance on the use of a treatment train as well as how to calculate the pollutant removal efficiency for storm water controls in series. Chapter 5 also provides guidance for choosing the appropriate storm water control(s) for a site as well as the basic considerations and limitations on the use of a particular storm water control.

Table 1.4.3-1 Suitability of Storm Water Controls to Meet *integrated* Design Approach

Category	*integrated* Storm Water Controls	Water Quality Protection	Streambank Protection	On-Site Flood Control	Downstream Flood Control
Bioretention Areas	Bioretention Areas	P	S	S	-
Channels	Enhanced Swales	P	S	S	S
	Channels, Grass	S	S	P	S
	Channels, Open	-	-	P	S
Chemical Treatment	Alum Treatment System	P	-	-	-
Conveyance Components	Culverts	-	-	P	P
	Energy Dissipation	-	P	S	S
	Inlets/Street Gutters	-	-	P	-
	Pipe Systems	-	P	P	P
Detention	Detention, Dry	S	P	P	P
	Detention, Extended Dry	S	P	P	P
	Detention, Multi-purpose Areas	-	P	P	P
	Detention, Underground	-	P	P	P
Filtration	Filter Strips	S	-	-	-
	Organic Filters	P	-	-	-
	Planter Boxes	P	-	-	-
	Sand Filters, Surface/Perimeter	P	S	-	-
	Sand Filters, Underground	P	-	-	-
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	S	-	-	-
Infiltration	Downspout Drywell	P	-	-	-
	Infiltration Trenches	P	S	-	-
	Soakage Trenches	P	S	-	-
Ponds	Ponds, Storm Water	P	P	P	P
Porous Surfaces	Green Roof	P	S	-	-
	Modular Porous Paver Systems	S	S	-	-
	Porous Concrete	S	S	-	-
Proprietary Systems	Proprietary Systems *	S	S	S	S
Re-Use	Rain Barrels	P	-	-	-
Wetlands	Wetlands, Storm Water	P	P	P	P
	Wetlands, Submerged Gravel	P	P	S	-

P = **Primary Control:** Able to meet design criterion if properly designed, constructed and maintained.

S = **Secondary Control:** May partially meet design criteria. May be a Primary Control but designated as a Secondary due to other considerations. For Water Quality Protection, recommended for limited use in approved community-designated areas.

- = Not typically used or able to meet design criterion.

***** = The application and performance of proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data.

References

Center for Watershed Protection (CWP), 1998. Better Site Design: A Handbook for Changing Development Rules in Your Community.

Delaware Department of Natural Resources and Environmental Control, 1997. Conservation Design for Storm Water Management. Prepared by the Brandywine Conservancy.

Marsh, W., 1983. Landscape Planning: Environmental Applications. New York: John Wiley & Sons.

Maryland Department of the Environment, 2000. Maryland Storm Water Design Manual, Volumes I and II. Prepared by Center for Watershed Protection (CWP).

Massachusetts Department of Environmental Protection / Massachusetts Office of Coastal Zone Management, 1997. Storm Water Management -- Volume One: Storm Water Policy Handbook, and Volume Two: Storm Water Technical Handbook.

Minnesota Pollution Control Agency, 1989. Protecting Water Quality in Urban Areas.

North Carolina Department of Environment and Natural Resources, 1998. Storm Water Management Site Planning.

Prince George's County, MD, 1999. Low-Impact Development Design Strategies, An Integrated Design Approach.

Schueler, T., 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. Metropolitan Washington Council of Governments.

Schueler, T., 1995. Site Planning for Urban Stream Protection. Prepared by Center for Watershed Protection (CWP). Metropolitan Washington Council of Governments.

Sykes, R.D., 1989. Chapter 31 – Site Planning. University of Minnesota.

Urban Land Institute, 1992. Density by Design.

Virginia Polytechnic Institute and State University, 2000. Water Sensitive Site Planning. College of Architecture and Urban Studies.

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CHAPTER

2

HYDROLOGIC ANALYSIS

Section 2.1
Estimating Runoff

2.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of storm water runoff. The analysis of these parameters is fundamental to the design of storm water management facilities, such as storm drainage systems and structural storm water controls. In the hydrologic analysis of a development/redevelopment site, there are a number of variable factors that affect the nature of storm water runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Rainfall abstraction rates (Initial and continued)
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods available to estimate runoff characteristics for a site or drainage subbasin; however, the following methods have been selected to support hydrologic site analysis for the design methods and procedures included in this Manual:

- Rational Method
- SCS Unit Hydrograph Method
- Snyder's Unit Hydrograph Method
- USGS & TXDOT Regression Equations
- iSWM Water Quality Protection Volume Calculation
- Water Balance Calculations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

Table 2.1.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 2.1.1-2 provides some limitations on the use of several methods.

In general:

The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.

The USGS (U.S. Geological Survey) and TXDOT (Texas Department of Transportation) regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. These equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate general regression equations might not be appropriate.

Table 2.1.1-1 Applications of the Recommended Hydrologic Methods

Method	Manual Section	Rational Method	SCS Method	Modified Rational	Snyder's Unit Hydrograph	USGS / TXDOT Equations	iSWM Water Quality Volume Calculation
Water Quality Protection Volume (WQ_v)	1.4						✓
Streambank Protection Volume (SP_v)	1.4		✓		✓		
Flood Control Discharge (Q_f)	1.4		✓		✓	✓	
Storage Facilities	4.5		✓	✓	✓		
Outlet Structures	4.6		✓		✓		
Gutter Flow and Inlets	3.2	✓					
Storm Drain Pipes	3.2	✓	✓		✓		
Culverts	4.2	✓	✓		✓	✓	
Bridges	4.3		✓		✓		
Small Ditches	4.4	✓	✓		✓		
Open Channels	4.4		✓		✓	✓	
Energy Dissipation	4.7		✓		✓		

Table 2.1.1-2 Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 – 200 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Modified Rational	0 – 200 acres	Method can be used for estimating runoff volumes for storage design.
Unit Hydrograph[2]	> 100 acres	Method can be used for estimating peak flows and hydrographs for all design applications.
TXDOT Regression Equations	10 to 100 mi^2	Method can be used for estimating peak flows for rural design applications.
USGS Regression Equations	3 – 40 mi^2	Method can be used for estimating peak flows for urban design applications.
iSWM Water Quality Protection Volume Calculation	Limits set for each Structural Control	Method used for calculating the Water Quality Protection Volume (WQ_v)

[1]Size limitation refers to the drainage basin for the storm water management facility (e.g., culvert, inlet).

[2]This refers to the SCS and Snyder's routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.

If other hydrologic methods are to be considered and used by a local review authority or design engineer, the method should first be calibrated to local conditions and tested for accuracy and reliability. If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

2.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 2.1.2-1 Symbols and Definitions

Symbol	Definition	Units
A	Drainage area or area	acres or square feet
B_f	Baseflow	acre-feet
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Time interval	hours
E	Evaporation	ft
E_t	Evapotranspiration	ft
F	Pond and swamp adjustment factor	-
G_h	Hydraulic gradient	-
I or i	Rainfall intensity	in/hr
I	Percent of impervious cover	%
I	Infiltration	acre-feet
I_a	Initial abstraction from total rainfall	in
k_h	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient	-
NRCS	Natural Resources Conservation Service (formerly SCS)	-
O_f	Overflow	acre-feet
P	Accumulated rainfall	in
P_2	2-year, 24-hour rainfall	in
P_w	Wetted perimeter	ft
PF	Peaking factor	-
Q	Rate of runoff	cfs (or inches)
Q_i	Peak inflow discharge	cfs
Q_o	Peak outflow discharge	cfs
Q_p	Peak rate of discharge	cfs
Q_{wq}	Water Quality peak rate of discharge	cfs
q	Storm runoff during a time interval	in
q_u	Unit peak discharge	cfs (or cfs/mi^2/inch)
R	Hydraulic radius	ft
R_o	Runoff	acre-feet
R_v	Runoff Coefficient	-
S	Ground slope	ft/ft or %
S	Potential maximum retention	in
S	Slope of hydraulic grade line	ft/ft
SCS	Soil Conservation Service (Now NRCS)	-
SP_v	Streambank Protection Volume	acre-feet
T	Channel top width	ft
T_L	Lag time	hours
T_p	Time to peak	hours
T_t	Travel time	hours
t	Time	min
t_c	Time of concentration	min

Table 2.1.2-1 Symbols and Definitions

Symbol	Definition	Units
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
V_d	Developed runoff volume	in
V_f	Flood control volume	acre-feet
V_r	Runoff volume	acre-feet
V_s	Storage volume	acre-feet
WQ_v	Water quality protection volume	acre-feet

2.1.3 Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs
Depth (inches) – Total amount of rainfall occurring during the storm duration
Intensity (inches per hour) – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedance probability* or *return period*.

Exceedance Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically in years
Return Period – Average length of time between events, which have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100 years.

Rainfall intensities for the 17 counties which participate in the NCTCOG area (see Figure 2.1.3-1) are provided in Appendix A and should be used for all hydrologic analysis within the given county. The values in these tables were derived in the following way:

- New IDF values for the 1-year through 500-year storm return periods were determined for the NCTCOG area on a county by county basis.
- All values were plotted and smoothed to ensure continuity. The values were smoothed by fitting an equation of the form:

  $$i = b/(t + d)^e \qquad (2.1.1)$$

  where *i* is inches per hour and *t* is the rainfall duration in minutes. The parameters *b, d and e* are found at the top of each of the tables in Appendix A.
- The tabular values in Appendix A were determined from the new IDF curves.

Figure 2.1.3-2 shows an example Intensity-Duration-Frequency (IDF) Curve for Dallas County, for seven storms (1-year – 100-year). These curves are plots of the tabular values. No values are given for times less than 5 minutes. The 500-year values are given for durations no less than 15 minutes.

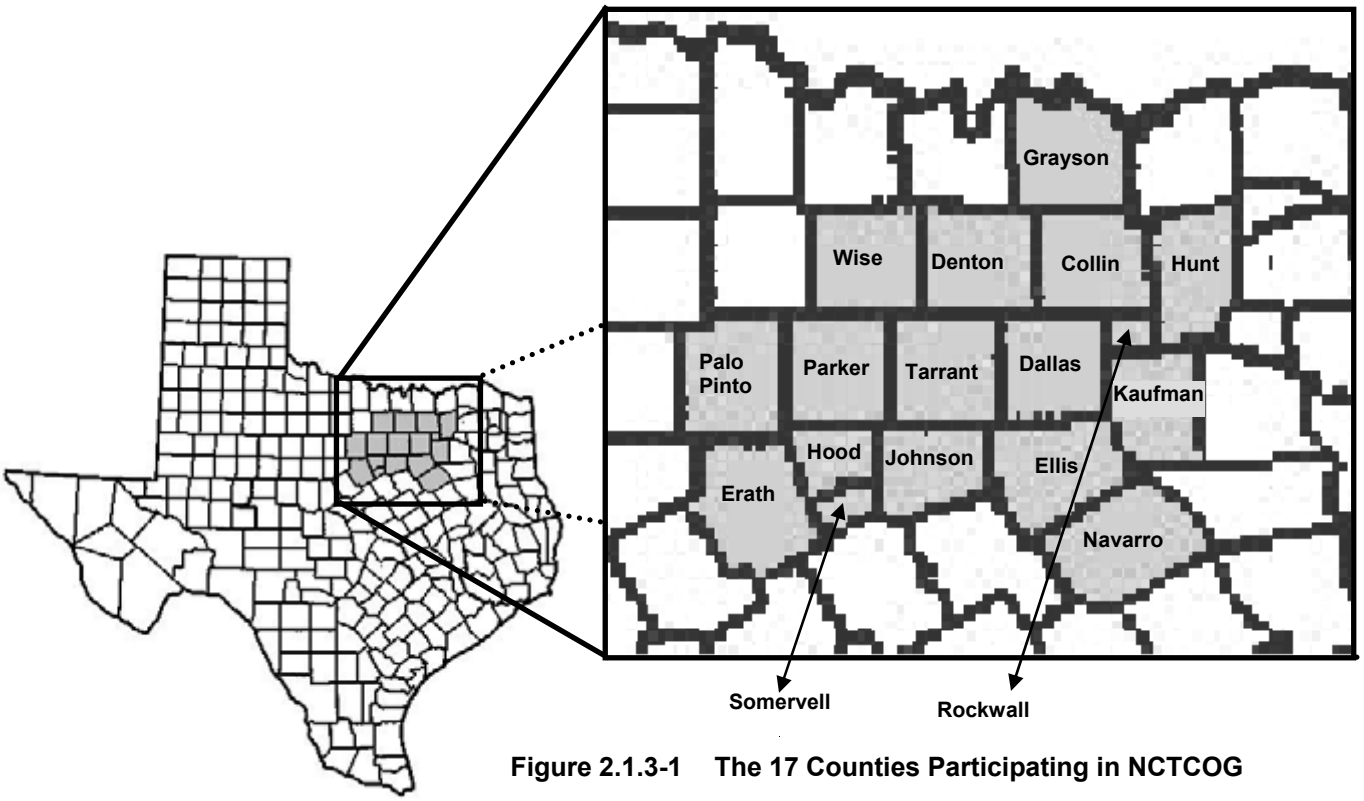


Figure 2.1.3-1 The 17 Counties Participating in NCTCOG

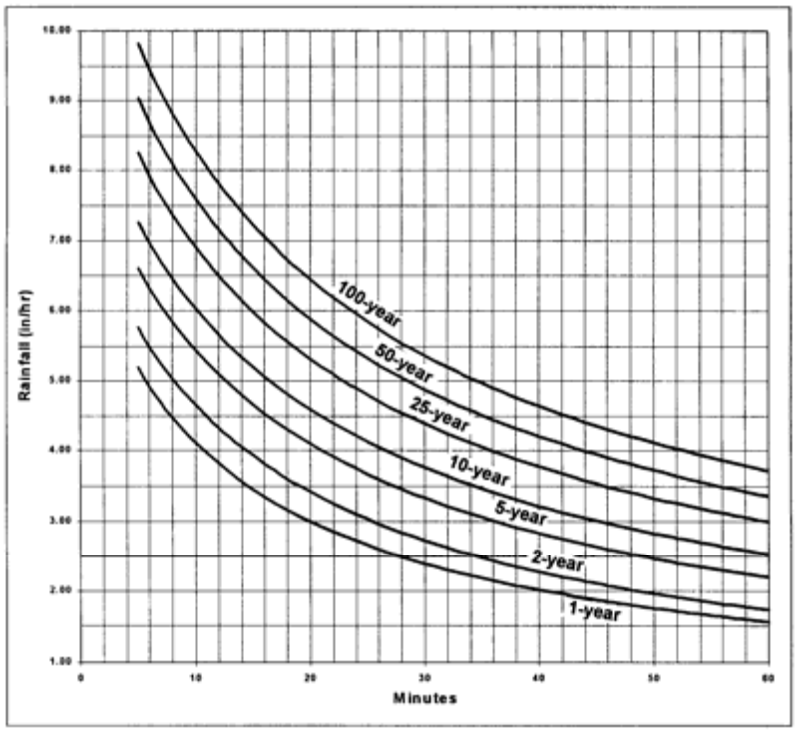


Figure 2.1.3-2 Example IDF Curve (Dallas County, Texas)

2.1.4 Rational Method

2.1.4.1 Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula adheres to the following assumptions:

- The predicted peak discharge has the same probability of occurrence (return period) as the rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate adjustments are appropriate.

2.1.4.2 Application

The Rational Method can be used to estimate storm water runoff peak flows for the design of gutter flows, drainage inlets, storm drainpipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 200 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, the Modified Rational method is used by some for design of small detention facilities, so the method has been included in Subsection 2.1.7. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in Subsection 2.1.7 corrects this deficiency in the method and can be used for detention design for drainage areas up to 200 acres.

The Rational Method should not be used for calculating peak flows downstream of bridges, culverts, or storm sewers that may act as restrictions causing storage, which impacts the peak rate of discharge.

2.1.4.3 Equations

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity for a duration equal to the time of concentration, t_c (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

$$Q = CIA \qquad (2.1.2)$$

where:

Q = maximum rate of runoff (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall
I = average rainfall intensity for a duration equal to the t_c (in/hr)
A = drainage area contributing to the design location (acres)

The coefficients given in Table 2.1.4-2 are applicable for storms with return periods less than or equal to 10 years. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f. The Rational Formula now becomes:

$$Q = C_f CIA \qquad (2.1.3)$$

The C_f values that can be used are listed in Table 2.1.4-1. The product of C_f times C shall not exceed 1.0.

Table 2.1.4-1 Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

2.1.4.4 Time of Concentration

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2.1.4-1 can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter. In urban areas, the length of overland flow distance should realistically be no more than 50 – 100 feet.

Although there is no formula for the graph shown in Figure 2.1.4-1, the formula often used, which seems to match the nomograph very closely, is as follows:

$$T_c = 1.8(1.1 - C)(D)^{0.5}/(S)^{(1/3)} \qquad (2.1.4)$$

where:

T_c = time of concentration (min)
C = average or composite runoff coefficient
D = distance from upper end of watershed to outlet (ft)
S = average slope along distance “D”, in percent (ft/100 ft)

Example: Given the following values, determine the time of concentration using (1) Equation 2.1.4, and (2) Figure 2.1.4-1: D = 250 ft, C = 0.7, S = 0.50% slope.

1. Figure 2.1.4-1 gives approximately 15 minutes.
2. $T_c = 1.8(1.1 – 0.7)(250)^{0.5}/(0.50)^{(1/3)} = 14.34$ min

Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow travel times that must be added as part of the overall time of concentration. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in Figure 2.1.4-2. More guidance on travel time estimation is given in Section 2.1.5.6.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes. Note that the time of concentration cannot be less than 5 minutes or that which is established by local standards.

The following table gives recommended minimum and maximum times of concentration based on land use categories. The minimum time of concentration should be used for the most upstream inlet (minimum inlet time). Computed downstream travel times will be added to determine times of concentration through the system. For anticipated future upstream development, the time of concentration should be no greater than the maximum.

Land Use	Minimum (minutes)	Maximum (minutes)
Residential Development	15	30
Commercial and Industrial	10	25
Central Business District	10	15

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

2.1.4.5 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in the rainfall tables in Appendix A.

2.1.4.6 Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2.1.4-2 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2.1.4-2 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients

for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

Table 2.1.4-2 Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficients (C)
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Agricultural	0.30
Forest	0.15
Streams, Lakes, Water Surfaces	1.00
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single Family (1/8 acre lots)	0.65
Single Family (1/4 acre lots)	0.60
Single Family (1/2 acre lots)	0.55
Single Family (1+ acre lots)	0.45
Multi-Family Units, (Light)	0.65
Multi-Family, (Heavy)	0.85
Commercial/Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover:	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

2.1.4.7 Example Problem

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

Site Data

From a topographic map of the City of Arlington (Tarrant County) and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition, the following data were measured:

Average overland slope = 2.0%

Length of overland flow = 50 ft

Length of main basin channel = 2,250 ft

Slope of channel = .018 ft/ft = 1.8%

Roughness coefficient (n) of channel was estimated to be 0.090

From existing land use maps, land use for the drainage basin was estimated to be:
Residential (single family – ¼ acre lots) - 80%
Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 2.1.4-2 to be 0.10.

Time of Concentration

From Figure 2.1.4-1 with an overland flow length of 50 ft, slope of 2% and a C of 0.10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 2.1.4-2 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

$$\text{Flow Time} = \frac{2{,}250 \text{ feet}}{(3.1 \text{ ft/s}) / (60 \text{ s/min})} = 12.1 \text{ minutes}$$

and t_c = 10 + 12.1 = 22.1 min (use 22 min)

Rainfall Intensity

From Table A-16 in Appendix A, using a duration equal to 22 minutes,

I_{25} (25-yr return period) = 5.41 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 2.1.4-2.

Land Use	Percent of Total Land Area	Runoff Coefficient	Weighted Runoff Coefficient
Residential			
(Single Family – ¼ acre lots)	0.80	0.60	0.48
Graded area	0.20	0.30	0.06
Total Weighted Runoff Coefficient = 0.54			

*Column 3 equals column 1 multiplied by column 2.

Peak Runoff

The estimate of peak runoff for a 25-yr design storm for the given basin is:

$Q_{25} = C_f CIA = (1.10)(.54)(5.41)(23) = 73.9$ cfs

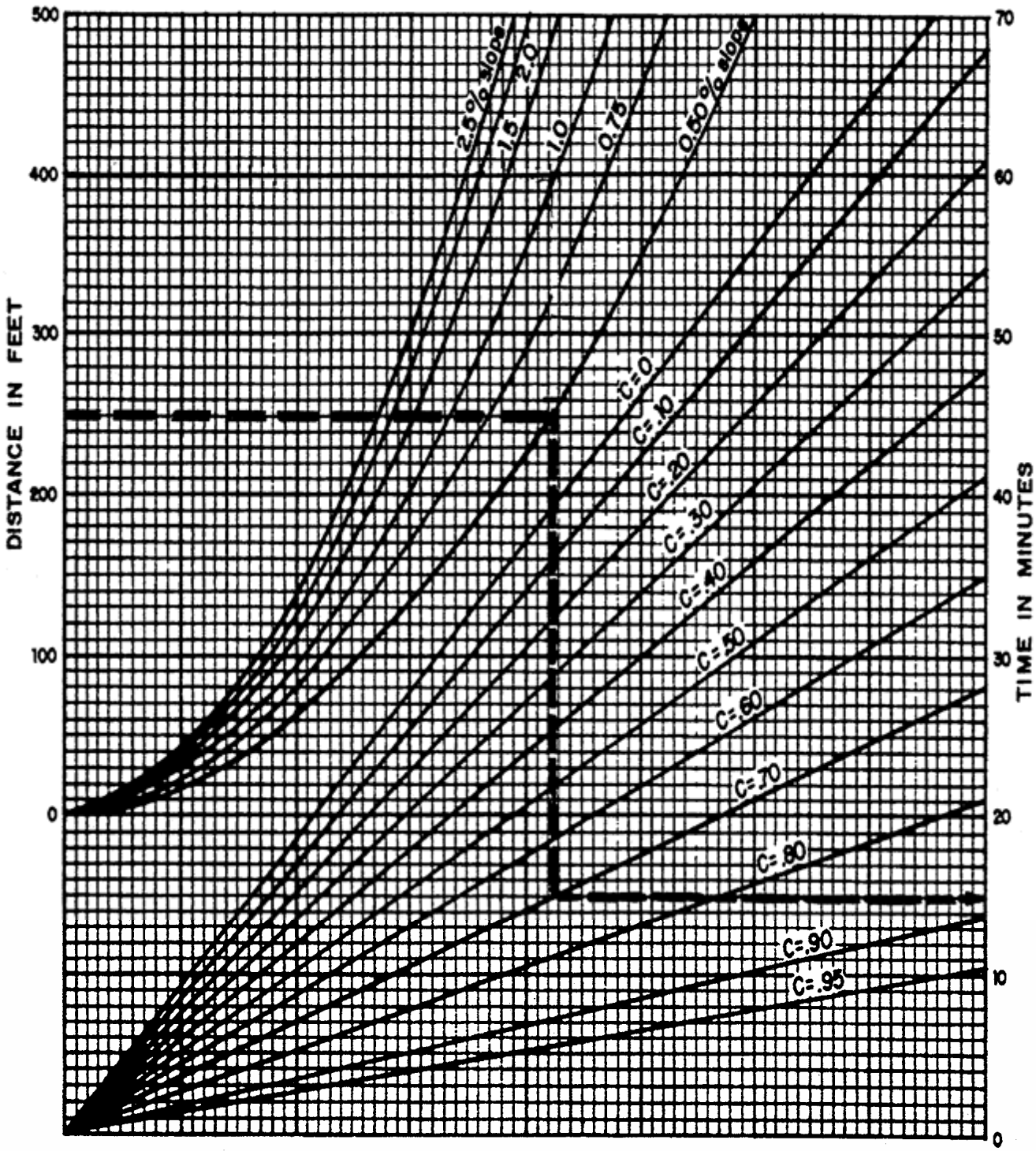
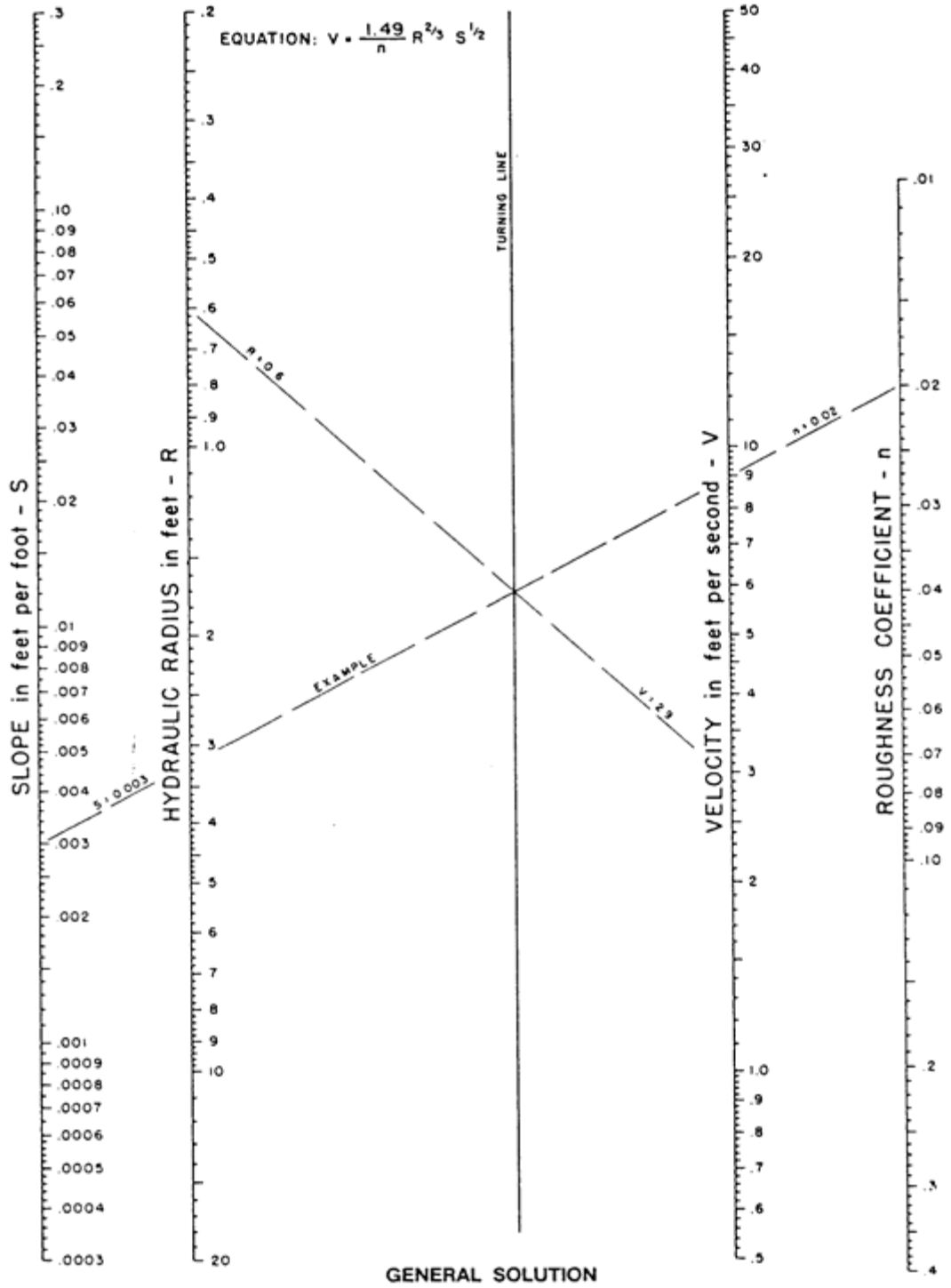


Figure 2.1.4-1 Rational Formula - Overland Time of Flow Nomograph
(Source: Airport Drainage, Federal Aviation Administration, 1965)



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 2.1.4-2 Manning's Equation Nomograph
(Source: USDOT, FHWA, HDS-3 (1961))

2.1.5 SCS Hydrologic Method

2.1.5.1 Introduction

The Soil Conservation Service[1] (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area.
2. Calculation of time of concentration to the study point.
3. Using the Type II rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 2.1.5-1 for the geographic boundaries for the different SCS rainfall distributions.
4. Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

2.1.5.2 Application

The SCS method can be used for both the estimation of storm water runoff peak rates and the generation of hydrographs for the routing of storm water flows. The simplified method of subsection 2.1.5.7 can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches, open channels, and energy dissipators.

2.1.5.3 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size, and slope are constant, the unit hydrograph approach assumes there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basic concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to North Central Texas is based on a storm event that has a Type II time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 2.1.5-1).

[1] The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

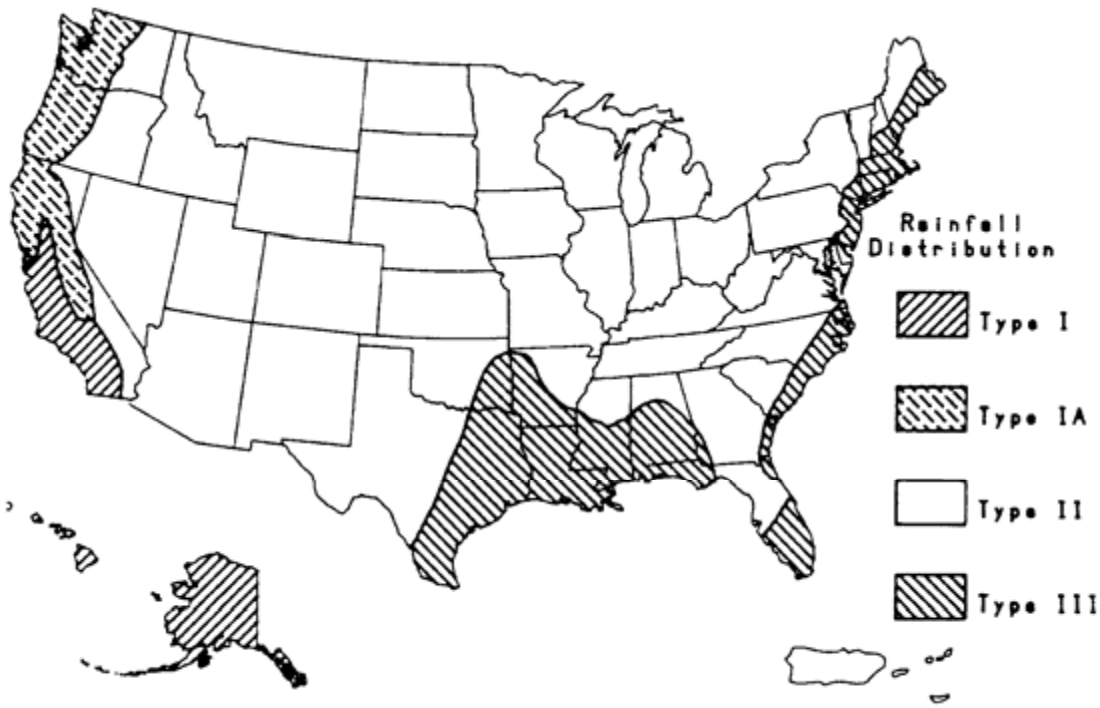


Figure 2.1.5-1 Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = (P - I_a)^2 / [(P - I_a) + S] \qquad (2.1.5)$$

where:

- Q = accumulated direct runoff (in)
- P = accumulated rainfall (potential maximum runoff) (in)
- I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
- S = 1000/CN - 10 where CN = SCS curve number

An empirical relationship used in the SCS method for estimating I_a is:

$$I_a = 0.2S \qquad (2.1.6)$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment. Table 2.1.5-3 provides values of I_a for a wide range of curve numbers (CN).

Substituting 0.2S for I_a in equation 2.1.5, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (2.1.7)$$

Figure 2.1.5-2 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a curve number of 85.

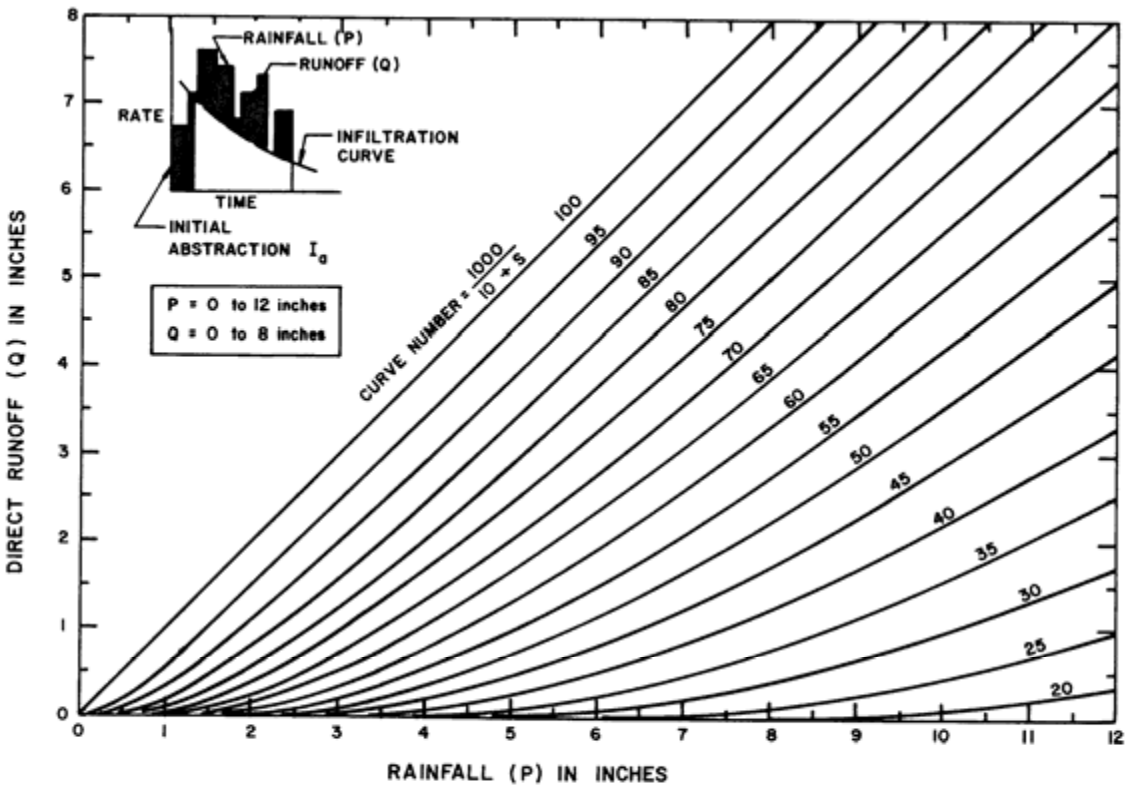


Figure 2.1.5-2 SCS Solution of the Runoff Equation
(Source: SCS, TR-55, Second Edition, June 1986)

Equation 2.1.6 can be rearranged so the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \qquad (2.1.8)$$

2.1.5.4 Runoff Factor (CN)

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups.

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Texas and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local USDA Natural Resources Conservation Service offices for use in estimating soil type. Appendix B contains hydrologic soils classification data for North Central Texas. County specific data can be found on-line through NRCS at http://soils.usda.gov/ and/or www.nctcog.dst.tx.us/.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 2.1.5-1 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

Composite Curve Number Calculation Example			
Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil Group B	0.80	0.85	0.68
Meadow Good condition Soil Group C	0.20	0.71	0.14
Total Weighted Curve Number = 0.68 + 0.14 = 0.82			

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

2.1.5.5 Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2.1.5-1 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over

pervious areas and then into a drainage system. It is possible for curve number values from urban areas to be reduced by not directly connecting impervious surfaces in the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

The CNs provided in Table 2.1.5-1 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. Pervious urban areas are equivalent to pasture in good hydrologic condition, and
2. Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2.1.5-1 are not applicable, use Figure 2.1.5-3 to compute a composite CN. For example, Table 2.1.5-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2.1.5-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Table 2.1.5-1 Runoff Curve Numbers[1]

Cover Description		Curve numbers for hydrologic soil groups			
Cover type and hydrologic condition	*Average percent impervious area[2]*	A	B	C	D
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93

Table 2.1.5-1 Runoff Curve Numbers[1]

Cover Description		Curve numbers for hydrologic soil groups			
Cover type and hydrologic condition	*Average percent impervious area[2]*	A	B	C	D
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

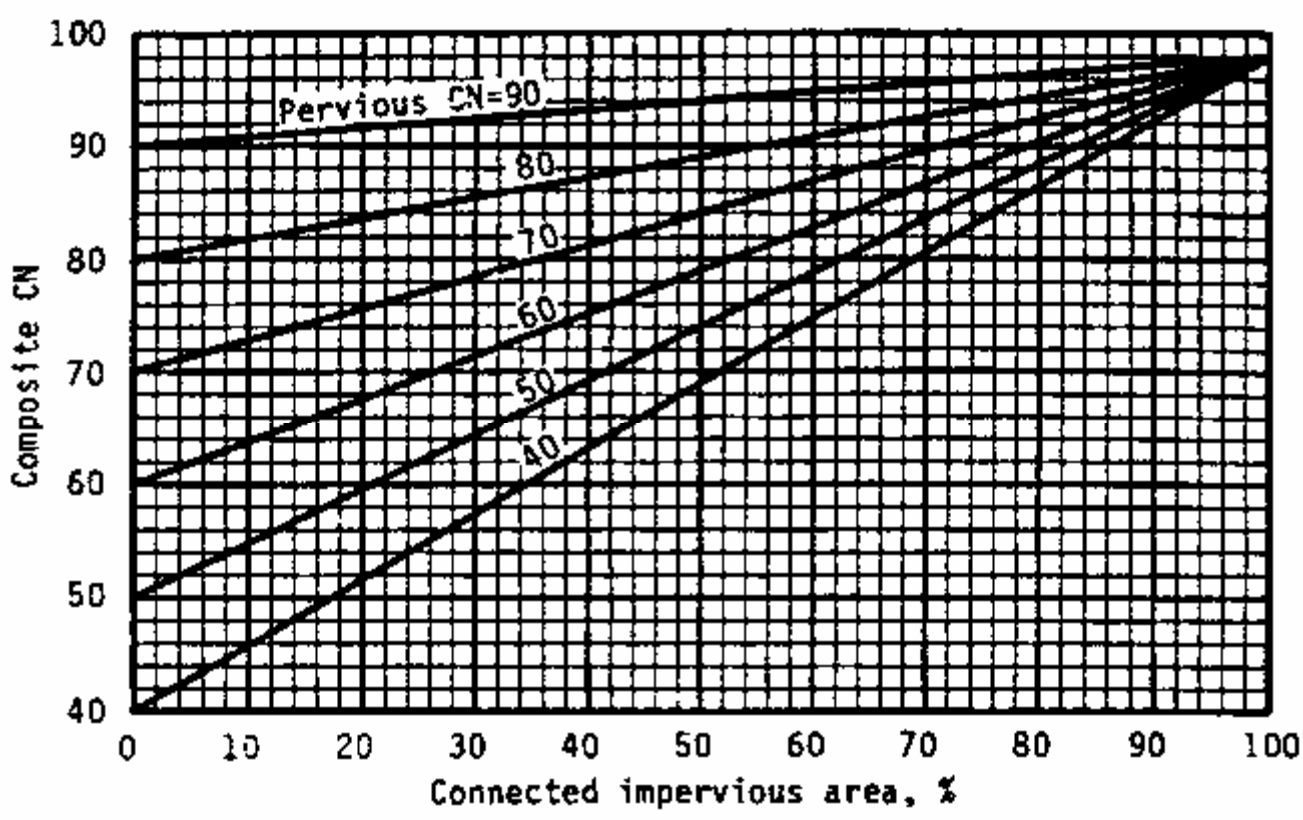


Figure 2.1.5-3 Composite CN with Connected Impervious Areas
(Source: SCS, TR-55, Second Edition, June 1986)

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2.1.5-4 if total impervious area is less than 30% or (2) use Figure 2.1.5-3 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When the impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 2.1.5-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 2.1.5-4 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2.1.5-3) would be 68.

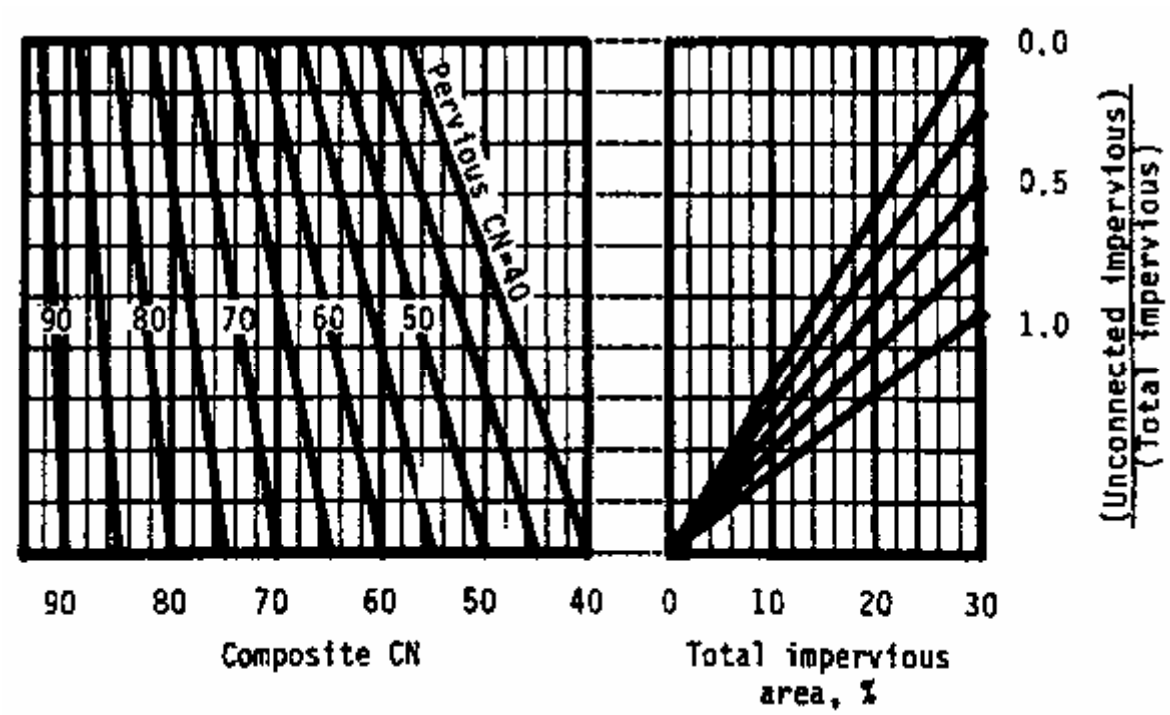


Figure 2.1.5-4 Composite CN with Unconnected Impervious Areas (Total Impervious Area Less Than 30%)
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.6 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/3600V \qquad (2.1.9)$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} \qquad (2.1.10)$$

where:

T_t = travel time (hr)
n = Manning roughness coefficient (see Table 2.1.5-2)
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall
S = land slope (ft/ft)

Table 2.1.5-2 Roughness Coefficients (Manning's n) for Sheet Flow[1]

Surface Description	n
Smooth surfaces	
(concrete, asphalt, gravel or bare soil)	0.011
Fallow	
(no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range	
(natural)	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information by Engman (1986).
[2] Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.
Source: SCS, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2.1.5-5, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2.1.5-5, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $V = 16.13(S)^{0.5}$ **(2.1.11)**

Paved $V = 20.33(S)^{0.5}$ **(2.1.12)**

where:

V = average velocity (ft/s)

S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 2.1.5-5 or equations 2.1.11 or 2.1.12, use equation 2.1.9 to estimate travel time for the shallow concentrated flow segment.

Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where stream designations appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is

$$V = (1.49/n)\ (R)^{2/3}\ (S)^{1/2} \qquad (2.1.13)$$

where:

V = average velocity (ft/s)

R = hydraulic radius (ft) and is equal to A/P_w

A = cross sectional flow area (ft^2)

P_w = wetted perimeter (ft)

S = slope of the hydraulic grade line (ft/ft)

n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 2.1.13, T_t for the channel segment can be estimated using equation 2.1.9.

Limitations

- Equations in this section should not be used for sheet flow longer than 50 feet for impervious surfaces.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

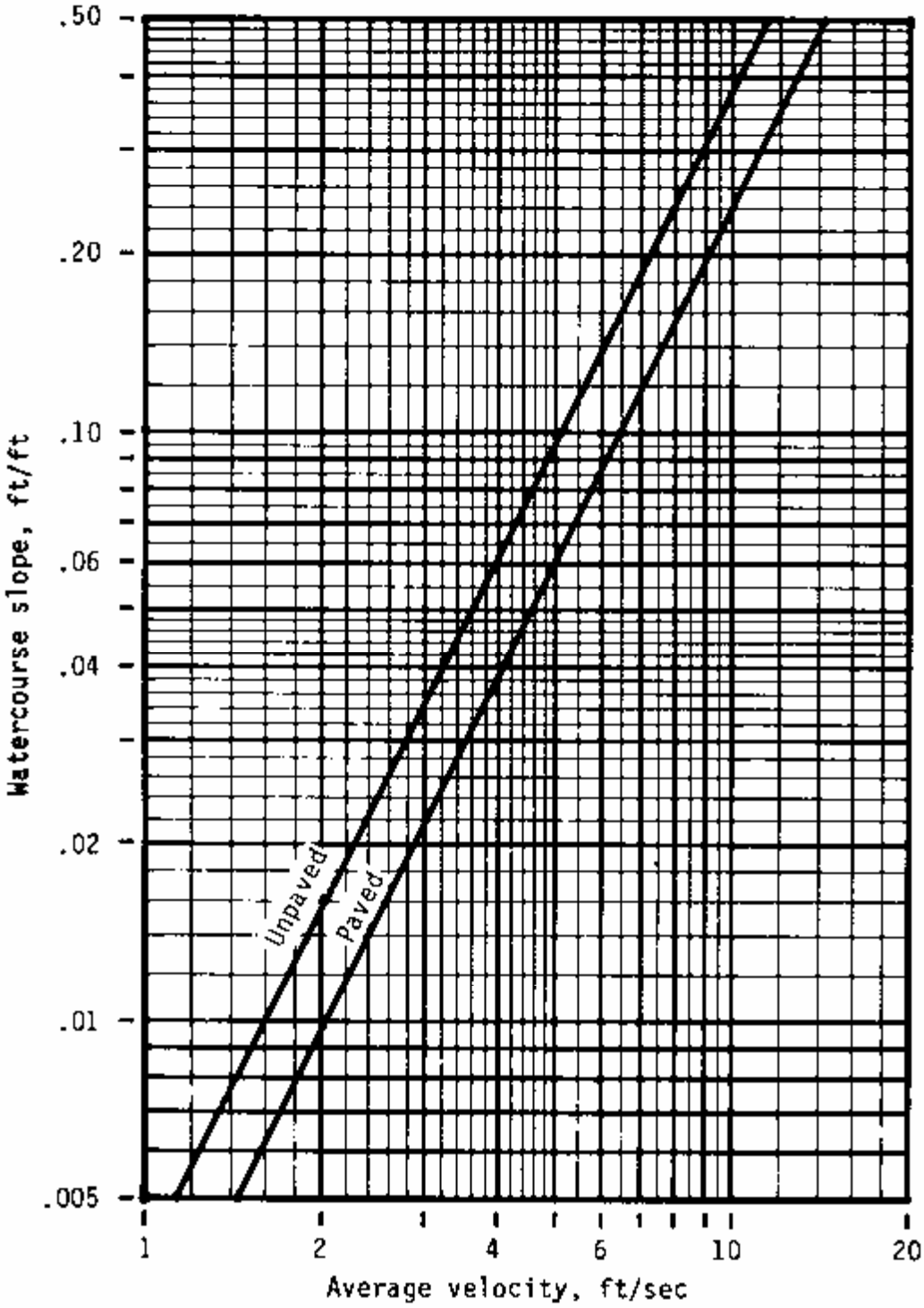


Figure 2.1.5-5 Average Velocities - Shallow Concentrated Flow
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.7 Simplified SCS Peak Runoff Rate Estimation

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses, which can be described by a single CN value. The peak discharge equation is:

$$Q_p = q_u A Q F_p \qquad (2.1.14)$$

where:

Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth (P) is determined from the rainfall tables in Appendix A for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 2.1.5-1 and direct runoff, Q, is calculated using equation 2.1.7.
3. The CN value is used to determine the initial abstraction, I_a, from Table 2.1.5-3, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
4. The watershed time of concentration is computed using the procedures in subsection 2.1.5.6 and is used with the ratio I_a/P to obtain the unit peak discharge (q_u) from Figure 2.1.5-6 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figure 2.1.5-6 is based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used. Peaking factors are discussed further in Section 2.1.5.9.
5. The pond and swamp adjustment factor, F_p, is estimated from below:

Pond and Swamp Areas (%*)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

6. The peak runoff rate is computed using equation 2.1.14.

Table 2.1.5-3 I_a Values for Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.74	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Source: SCS, TR-55, Second Edition, June 1986

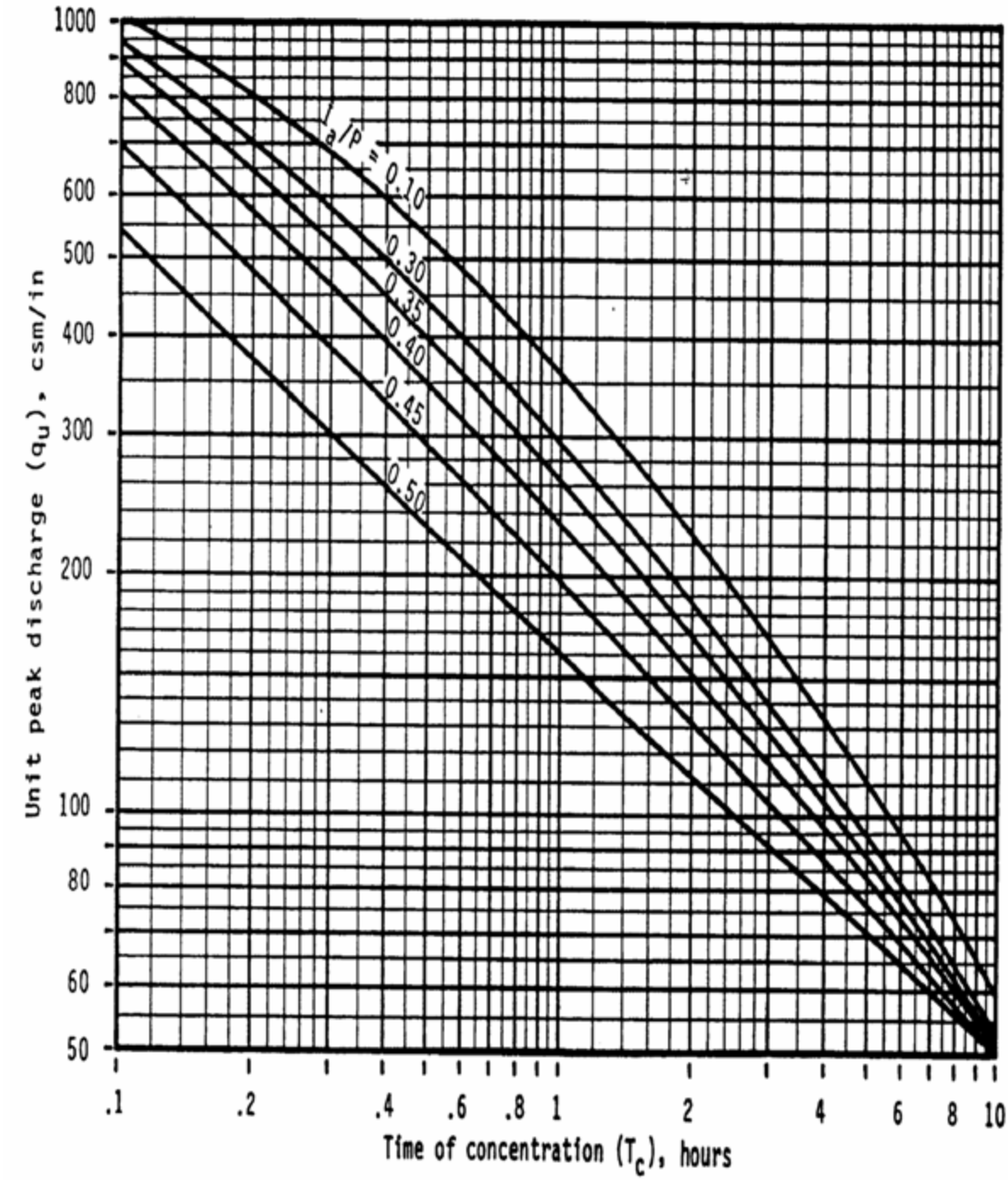


Figure 2.1.5-6 SCS Type II Unit Peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.8 Example Problem 1

Compute the 100-year peak discharge for a 50-acre watershed located in Fort Worth, which will be developed as follows:

1. Pasture / range land - good condition (hydrologic soil group D) = 10 ac
2. Pasture / range land - good condition (hydrologic soil group C) = 10 ac
3. 1/3 acre residential (hydrologic soil group D) = 20 ac
4. Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

Computations

1. Calculate rainfall excess:
   - The 100-year, 24-hour rainfall is 9.12 inches (.38 in/hr x 24 hours – From Appendix A, Table A-16).
   - Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	0.20	80	18.2
2	10 ac.	0.20	74	14.8
3	20 ac.	0.40	86	34.4
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00		83

   * from Equation 2.1.7 Q (100-year) = 7.1 inches

2. Calculate time of concentration

   The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.0
2	Shallow channel (unpaved)	750	1.7
3	Main channel*	1100	0.50

   * For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

   Segment 1 - Travel time from equation 2.1.10 with P_2 = 3.36 inches
   (0.14 x 24 – Appendix A, Table A-16)

   $T_t = [0.42(0.24 \text{ X } 40)^{0.8}] / [(3.36)^{0.5} (.020)^{0.4}] = 6.69$ minutes

   Segment 2 - Travel time from Figure 2.1.5-5 or equation 2.1.11
   V = 2.1 ft/sec (from equation 2.1.11)
   T_t = 750 / 60 (2.1) = 5.95 minutes

   Segment 3 - Using equation 2.1.13
   V = $(1.49/.06) (1.43)^{0.67} (.005)^{0.5}$ = 2.23 ft/sec
   T_t = 1100 / 60 (2.23) = 8.22 minutes

   t_c = 6.69 + 5.95 + 8.22 = 20.86 minutes (.35 hours)

3. Calculate I_a/P for CN = 83 (Table 2.1.5-1), I_a = .410 (Table 2.1.5-3)

   I_a/P = (.410 / 9.12) = .05

(Note: Use I_a/P = .10 to facilitate use of Figure 2.1.5-6.)

4. Unit discharge q_u (100-year) from Figure 2.1.5-6 = 650 csm/in
5. Calculate peak discharge with F_p = 1 using equation 2.1.14

   Q_{100} = 650 (50/640)(7.1)(1) = 360 cfs

2.1.5.9 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas.

A value of 484 should be used for most areas of North Texas; however, there are flat areas where a lesser value may be appropriate.

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand calculation. For that reason, only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other “administrative” parameters, which are peculiar to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

1. Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in subsection 2.1.5.3 is used. This storm is recommended for use in North Central Texas.
2. Development of curve numbers and lag times for the watershed using the methods described in subsections 2.1.5.4, 2.1.5.5, and 2.1.5.6.
3. Development of a unit hydrograph using the standard (peaking factor of 484) dimensionless unit hydrograph. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 2.1.7).
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called “convolution”).
6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 484, Figure 2.1.5-7 and Table 2.1.5-4 have been developed. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but, typically, should not be used for areas in North Central Texas.

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$q_u = (PF\ A) / (T_p) \qquad (2.1.15)$$

where:

q_u = unit hydrograph peak rate of discharge (cfs)

PF = peaking factor (484)

A = area (mi^2)

d = rainfall time increment (hr)

T_p = time to peak = d/2 + 0.6 t_c (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by the equation:

$$\frac{q}{q_u} = \left(\frac{t}{T_p}\ e^{[1-(t/T_p)]}\right)^X \qquad (2.1.16)$$

where X is 3.79 for the PF=484 unit hydrograph.

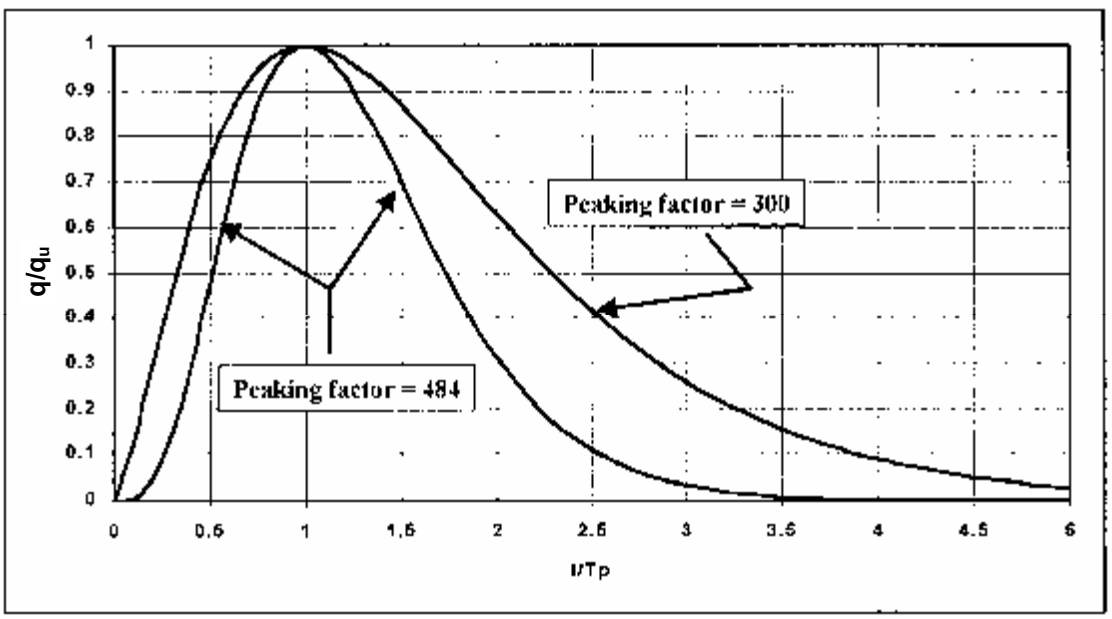


Figure 2.1.5-7 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 2.1.5-4 Dimensionless Unit Hydrograph With Peaking Factor of 484

	484	
t/T_t	q/q_u	Q/Q_p
0.0	0.0	0.0
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

2.1.5.10 Example Problem 2

Compute the unit hydrograph for the 50-acre watershed in example 2.1.5.8.

Computations

7. Calculate T_p and time increment

   The time of concentration (t_c) is calculated to be 20.86 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

   $T_p = d/2 + 0.6t_c = 3/2 + 0.6 * 20.86 = 14.02$ minutes (0.234 hrs)

8. Calculate q_{pu}

   $q_u = PF\ A/T_p = (484 * 50/640) / (0.234) = 162$ cfs

9. Calculate unit hydrograph.

   Based on spreadsheet calculations using equations 2.1.15 and 2.1.16, the table below has been derived.

Time		484	
t/T_p	time (min)	q/q_u	Q
0	0	0	0.00
0.21	3	0.06	9.23
0.43	6.0	0.35	56.77
0.64	9.0	0.72	117.29
0.86	12.0	0.96	155.09
1.00	14.02	1.00	162.00
1.07	15.0	0.99	160.57
1.28	18.0	0.88	142.42
1.50	21.0	0.70	113.52
1.71	24.0	0.52	83.69
1.93	27.0	0.36	58.12
2.14	30.0	0.24	38.51
2.35	33.0	0.15	24.56
2.57	36.0	0.09	15.18
2.78	39.0	0.06	9.14
3.00	42.0	0.03	5.38
3.21	45.0	0.02	3.10
3.42	48.0	0.01	1.76
3.64	51.0	0.01	0.99
3.85	54.0	0.00	0.54
4.07	57.0	0.00	0.30
4.28	60.0	0.00	0.16
4.49	63.0	0.00	0.09
4.71	66.0	0.00	0.05
4.92	69.0	0.00	0.02
5.14	72.0	0.00	0.01
5.35	75.0	0.00	0.01
5.56	78.00	0.00	0.00

2.1.5.11 Hydrologic Stream Routing

Water requires a certain amount of time to travel down a stream or channel reach. A flood wave is attenuated by friction and channel storage as it passes through the reach. The process of computing the travel time and attenuation of water flowing in the reach is often called routing.

Hydrologic routing involves the balancing of inflow, outflow, and volume of storage through the use of the continuity equation. The relation between the outflow rate and storage in the system is also required.

Travel time and attenuation characteristics vary widely between different streams. The travel time is dependent on characteristics such as length, slope, friction, and flow depth. Attenuation is also dependent on friction, in addition to other characteristics such as channel storage. Many routing methods have been developed under different assumptions and for different stream types. Some of the routing methods include: kinematic wave, lag, modified Puls, Muskingum, Muskingum-Cunge 8-point section, and Muskingum-Cunge standard section.

The routing methods selected for use in North Central Texas are the Modified Puls and the Muskingum-Cunge methods (USACE, HEC-HMS, 2000 and Bedient and Huber, 1988).

2.1.6 Snyder's Unit Hydrograph Method

2.1.6.1 Introduction

Snyder's unit hydrograph method is the primary method utilized by the Corps of Engineers Fort Worth District for the majority of hydrologic studies in the region, and is also commonly used by consultants and other entities within the NCTCOG region. It is similar in nature to the SCS method, in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm.

2.1.6.2 Application

Snyder's unit hydrograph method may be used for drainage areas 100 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations* and The Bureau of Reclamation's "Flood Hydrology Manual, A Water Resources Technical Publication," utilizes the following equations:

$$t_p = C_t (L L_{ca})^{0.3} \quad (2.1.17)$$

$$t_r = t_p \div 5.5 \quad (2.1.18)$$

$$q_p = C_p 640 \div t_p \quad (2.1.19)$$

$$t_{pR} = t_p + 0.25(t_R - t_r) \quad (2.1.20)$$

$$q_{pR} = C_p 640 \div t_{pR} \quad (2.1.21)$$

$$q_{pR} = q_p t_p \div t_{pR} \quad (2.1.22)$$

$$Q_p = q_p A \quad (2.1.23)$$

The terms in the above equations are defined as:

t_r = The standard unit rainfall duration, in hours.

t_R = The unit rainfall duration in hours other than standard unit, t_r, adopted in specific study.

t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.

t_{pR} = The lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph in hours.

q_p = The peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cfs/sq. mi.

q_{pR} = The peak rate of discharge in cfs/sq mi. of unit in hydrograph for unit rainfall duration, t_R.

Q_p = The peak rate of discharge of unit hydrograph in cfs.
A = The drainage area in square miles.
L_{ca} = The river mileage from the design point to the centroid of gravity of the drainage area.
L = The river mileage from the given station to the upstream limits of the drainage area.
C_t = Coefficient depending upon units and drainage basin characteristics.
C_p = Coefficient depending upon units and drainage basin characteristics.

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t range from 0.4 to 2.3 and average about 1.1. The value of C_t for the East Fork Trinity River is 2.0. C_t for a watershed can be estimated if the lag time, t_p, stream length, L, and distance to the basin centroid, L_{ca}, are known. The coefficient C_p is the peaking coefficient, which typically ranges from 0.3 to 1.2 with an average value of 0.8, and is related to the flood wave and storage conditions of the watershed. The C_p value for the East Fork Trinity River is 0.69. Larger values of C_p are generally associated with smaller values of C_t. Typical values of C_p are listed in Table 2.1.6-1.

Table 2.1.6-1 Typical Values of C_p

Typical Drainage Area Characteristics	Value of C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

2.1.6.3 Urbanization Curves

To account for the effects of urbanization, another method was developed by the Corps of Engineers to adjust the t_p coefficient. Urbanization curves allow for the determination of t_p based on the percent urbanization and the type of soil in the study area. Urbanization curves for the Dallas-Fort Worth area were determined from the equation below:

$$t_p = 10\text{^}[0.3833*\log_{10}(L*L_{ca}/(S_{st})\text{^}.5)+(\log_{10}(Ip))-BW * (\%Urb/100)] \quad \textbf{(2.1.24)}$$

$$S_{st} = (el_{85\%} - el_{15\%})/(0.7*L) \quad \textbf{(2.1.25)}$$

where:

t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.
L_{ca} = The river mileage from the design point to the centroid of the drainage area.
L = The river mileage from the design point to the upstream limits of the drainage area.
S_{st} = The weighted slope of the flow path (ft/mi)
Ip = The calibration point, defined as t_p where ($L*L_{ca}/S_{st}$^.5) = 1 and urbanization = 0%.
BW = The bandwidth, equal to the log of the width between each 20% urbanization line.

%Urb = A value representative of the degree to which urbanization has occurred in the basin, in percent.

$el_{85\%}$ = The elevation at a location 85% upstream of the given station.

$el_{15\%}$ = The elevation at a location 15% upstream of the given station.

For the Dallas-Fort Worth area, the Ip values used are 0.94 for clay and 1.76 for sand. The bandwidth (BW) value for both of the soil types is 0.266. For a study area that is composed of both sand and clay, a weighted average of the two can be calculated by:

t_p weighted = % sand*t_p sand + % clay * t_p clay.

Design runoff may be determined for a given watershed by applying the intensity-duration-frequency relationships to the unit hydrograph by multiplying each ordinate of the unit hydrograph by the rainfall intensity.

2.1.6.4 Determination of Percent Urbanization and Percent Sand

The lag time, t_p, is the critical parameter in establishing the timing of the response of a watershed to rainfall. The degree of urbanization is an important variable that determines the value of the lag time. Thomas L. Nelson, Fort Worth District, USACE, defined the general relationship between the lag time, t_p, and the percent of Urbanization, %Urb, and presented a set of Urbanization Curves for the Dallas-Fort Worth area in 1970.

The soil type of a watershed also plays an important role in its response to rainfall. It was found that predominantly sandy soils responded differently to rainfall than predominantly clayey soils. Therefore, two sets of Urbanization Curves were developed to better define the lag time, one set for sandy soils and one set for clayey soils. A paper by Paul K. Rodman, Fort Worth District, USACE presented urbanization curves in 1977 for both "clay loam" and "clay" in the Fort Worth-Dallas area and other Texas locations.

To obtain consistency of computational results, it is necessary to have a logical and routine procedure for the determination of Percent Urbanization (%Urb) and Percent Sand/Clay (%Sand/%Clay). Procedures for their determination are presented below.

<u>Percent Urbanization</u>

Urbanization is defined as the percentage of the basin which has been developed and improved with channelization and/or a storm water collection network. Urbanization of natural and agricultural land converts pervious soils to impervious surfaces. Disturbed soils exhibit a lower infiltration capacity than natural soils. This results in less infiltration which translates to an increased volume of runoff.

Natural flow paths in the watershed may be replaced with prismatic channels. Significant drainage infrastructure may be added in a development composed of streets and gutters, storm sewers, open channels, and other drainage elements. This alteration of the original drainage system changes the watershed's response to precipitation. The addition of drainage infrastructure along with the increase in imperviousness results in significantly increased peak discharges and a greater volume of runoff.

The determination of the percent urbanization (%Urb) as used in the Urbanization Curves defined by Equation 2.1.24 is somewhat subjective, but is related to the type and intensity of development. The U.S. Army Corps of Engineers (USACE) has worked over the years to define the relationship between the type of development and the degree of urbanization. The result of their effort is reflected in Table 2.1.6-2. These are provided for the user's consideration and guidance.

Other techniques to relate the impacts of urbanization on rainfall runoff have been used. Another such technique is presented in Section 2.1.8.2 in the application of USGS regression equations to determine peak flows for urban basins.

Table 2.1.6-2 Percent Urbanization and Imperviousness Summary with Associated Land Use Categories

Land Use	Description	Percent Imperviousness	Percent Urbanization
Low Density Residential	Single family: ½ – 2 units per acre; average 1 unit per acre.	25	30
Medium Density Residential	Single family: 2 – 3½ units per acre; average 3 units per acre.	41	80
High Density Residential	Single family: greater than 3½ units per acre; average 4 units per acre.	47	90
Multifamily Residential	Row houses, apartments, townhouses, etc.	70	95
Mobile Home Parks	Single family: 5–8 units per acre.	20	40
Central Business District	Intensive, high-density commercial	95	95
Strip Commercial	Low-density commercial; average 3 units per acre.	90	90
Shopping Centers	Grocery stores, drug stores, malls, etc.	95	95
Institutional	Schools, churches, hospitals, etc.	40	50
Industrial	Industrial centers and parks; light and heavy industry.	90	95
Transportation	Major highways, railroads.	35	80
Communication	Microwave towers, etc.	35	50
Public Utilities	Transformer stations, transmission line right-of-way, sewage treatment facilities, water towers, and water treatment facilities.	60	70
Strip Settlement	Densities less than ½ – 2 units per acre; average 1 unit per 3 – 5 acres.	10	20
Parks and Developed Open Space	Parks, cemeteries, etc.	6	10
Developing	Land currently being developed.	15	20
Cropland		3	5
Grassland	Pasture, short grasses.	0	0
Woodlands, Forest		0	0
Water Bodies	Lakes, large ponds.	100	100
Barren Land	Bare exposed rock, strip mines, gravel pits.	0	0

Sources: Determination of Percent Urbanization/Imperviousness in Watersheds, May 1, 1986, U.S. Army Corps of Engineers
SCS, TR-55, Second Edition, June 1986

Percent Sand/Clay

The Fort Worth District, USACE, evaluated methods for determining the percent sand in a watershed and concluded that the permeability rate method was the best method. The procedure was described in the referenced report as follows.

"The permeability rate method uses the range of permeabilities found in the table of physical and chemical properties in the SCS soil surveys for multiple soil classifications and assigns a percent sand to each of the seven ranges. A percent sand of 0 is given to any soil with a permeability less than 0.06

inches per hour which corresponds to the permeability of the Houston Blackland clay upon which the clay urban curves are based. Also, a percent sand of 100 is given to any soil with a rate of 0.6 to 2.0 inches per hour which corresponds to the Crosstell series soil upon which the sandy loam curves are based. The percent sand for the permeability ranges 0.06 to 0.2 inches, 0.2 to 0.6 inches, 2.0 to 6.0 inches, 6.0 to 10.0 inches, and greater than 20 inches are 33, 66, 133, 166, 200 percent sand, respectively. Each soil in the watershed is assigned a percent sand based upon its permeability and a weighted average is computed." (USACE, 1986).

Table 2.1.6-3 Permeability Rating for the Determination of Percent Sand

Permeability (inches/hr)	Percent Sand Assignment (%)
< 0.06	0
0.06 to 0.20	33
0.20 to 0.60	66
0.60 to 2.00	100
2.00 to 6.00	133
6.00 to 20.00	166
> 20.00	200

The Houston Black soil series consists of moderately well-drained, deep, cyclic, clayey soils on wetlands. This series formed in alkaline, marine clay, and material weathered from shale. Land slopes range from 1 to 4 percent. The permeability is less than 0.06 inches per hour. This soil is the predominate series found in watersheds used to develop the Dallas-Fort Worth Clay Urbanization Curves. Therefore this soil has a percent sand of 8 for use with the urban curves. The Crosstell soil series consists of moderately well-drained, deep loamy soils on uplands that formed in shaley and clayey sediment containing thin strata of weakly cemented sandstone. Land slopes range from 1 to 6 percent. The permeability for this soil is in the range between 0.6 and 2.0 inches per hour. The Crosstell series is the major soil contained in watersheds used to derive the Dallas-Fort Worth Sandy Loam Urbanization Curves. This soil, therefore, has a percent sand of 100 for use with the urban curves.

Example: Procedure for the Determination of Percent Sand (%Sand).

Given the percent sand assignments below, determine the percent sand for Watershed B.

Watershed	Soil Type No.	Percent Sand	% of Area	% Sand * % Area
B	13	66	2.6	171.6
	23	33	39.7	1310.1
	32	133	31.4	4176.2
	51	33	1.7	56.1
	64	133	17.9	2380.7
	85	33	6.7	221.1
			100	8315.8

Weighted %Sand = 8315.8/100 = 83.2%

There is the possibility of computing greater than 100 percent sand for areas that are very sandy. Soil disturbances during development (urbanization) usually diminish the natural permeability of the soil. Often there is no data reflecting the permeability rate for an urban soil. Therefore, care should be used in applying this method. The percent sand assignment should be that of the controlling sublayer of the soil profile. Consideration should also be given to other factors affecting the initial and time rates of rainfall abstractions. For example, well-vegetated clayey soils may respond hydrologically more like a sandy soil.

Urban lands are usually taken one step down (lower percent sand) from soil types shown in the SCS soil report. The engineer should evaluate all factors bearing on the soil response and determine whether there is a need to make adjustments.

Loss Rates

Several loss rate methodologies are acceptable for use with the Snyder's Unit Hydrograph Method including:

- Block and Uniform
- Holtan
- SCS Curve Number
- Green and Ampt
- Exponential

Block and uniform loss rates developed by the Corps of Engineers during the development of the urbanization curves are listed by clay and sand categories. Losses for a specific basin are determined by a weighting procedure. Adjustments to these values are allowed based on historic storm reproductions.

Table 2.1.6-4 Hydrologic Loss Rates

Frequency	Losses			
	Clay		Sand	
	Block (in)	Uniform (in/hr)	Block (in)	Uniform (in/hr)
2-year	1.5	0.20	2.1	0.26
5-year	1.3	0.16	1.8	0.21
10-year	1.12	0.14	1.5	0.18
25-year	0.95	0.12	1.3	0.15
50-year	0.84	0.1	1.1	0.13
100-year	0.75	0.07	0.9	0.10

Stream Routing

The Modified Puls and Muskingum-Cunge are acceptable routing methods. See Section 2.1.5.11 for an explanation of routing methods and references for further information.

2.1.7 The Modified Rational Method

2.1.7.1 Introduction

For drainage areas of *less than 200 acres*, a modification of the Rational Method can be used for the estimation of storage volumes for detention calculations.

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. There are many variations on the approach. Figure 2.1.7-1 illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (t_c). An allowable target outflow is set (Q_a) based on pre-development conditions. The storm duration is t_d, and is varied until the storage volume (shaded gray area) is maximized. It is normally an iterative process done by hand or on a spreadsheet. Downstream analysis is not possible with this method as only approximate graphical routing takes place.

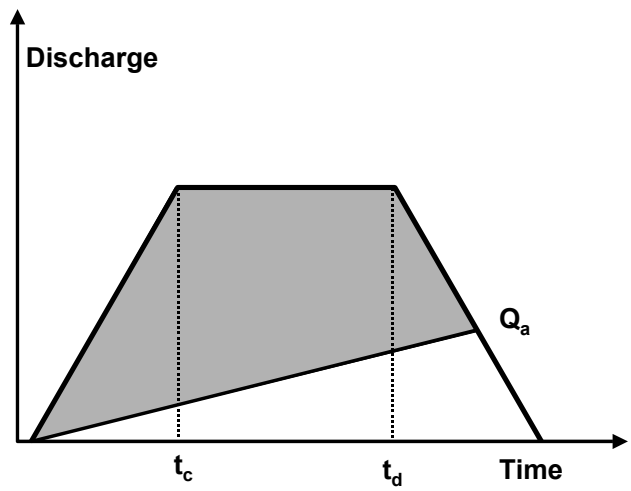


Figure 2.1.7-1 Modified Rational Definitions

2.1.7.2 Design Equations

The design of detention using the Modified Rational Method is presented as a non-iterative approach suitable for spreadsheet calculation (Debo & Reese, 2003).

The allowable release rate can be determined from:

$$Q_a = C_a\, i\, A \qquad (2.1.26)$$

where:

Q_a = allowable release rate (cfs)
C_a = predevelopment Rational Method runoff coefficient
i = rainfall intensity for the corresponding time of concentration (in/hr)
A = area (acres)

The critical duration of storm, the time value to determine rainfall intensity, at which the storage volume is maximized, is:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \qquad (2.1.27)$$

where:

T_d = critical storm duration (min)
Q_a = allowable release rate (cfs)
C = developed condition Rational Method runoff coefficient
A = area (acres)
a, b = rainfall factors dependent on location and return period taken from Table 2.1.7-1

The required storage volume, in cubic feet can be obtained from the equations below:

$$V_{preliminary} = 60\,[CAa - (2CabAQ_a)^{1/2} + (Q_a/2)\,(b-t_c)] \qquad (2.1.28a)$$

$$V_{max} = V_{preliminary} * P_{180}/P_{td} \qquad (2.1.28b)$$

where:

$V_{preliminary}$ = preliminary required storage (ft^3)

V_{max} = required storage (ft^3)
t_c = time of concentration for the developed condition (min)
P_{180} = 3-hour (180-minute) storm depth (in)
P_{td} = storm depth for the critical duration (in)
all other variables are as defined above

The equations above include the use of an adjustment factor to the calculated storage volume to account for under sizing. The factor (P_{180}/P_{td}) is the ratio of the 3-hour storm depth for the return frequency divided by the rainfall depth for the critical duration calculated in equation 2.1.27.

The Modified Rational Method also often under sizes storage facilities in flat and more sandy areas where the target discharge may be set too large, resulting in an oversized orifice. In these locations modifications to the C factor or time of concentration should be considered in the design of the orifice.

Table 2.1.7-1 Rainfall Factors "a" and "b" for the Modified Rational Method (1-year through 100-year return periods)

County		Return Interval						
		1	2	5	10	25	50	100
Collin	a	101.14	129.51	177.49	209.08	250.52	283.13	320.81
	b	14.214	16.634	20.174	21.668	22.821	23.455	24.502
Dallas	a	99.8	128.85	178.58	210.73	253.77	288.56	327.75
	b	14.114	16.624	20.352	21.785	23.03	23.866	24.893
Denton	a	97.258	124.47	173.1	205.74	248.54	283.99	325.18
	b	13.788	16.121	19.754	21.358	22.615	23.508	24.822
Ellis	a	101.94	129.3	181.43	214.61	259.34	295.76	336.3
	b	14.511	16.697	20.792	22.384	23.744	24.681	25.818
Erath	a	90.53	113.9	159.31	189.97	228.79	260.81	298.07
	b	13.32	14.99	18.439	19.981	20.955	21.65	22.712
Grayson	a	100.87	128.89	175.74	208.17	250.17	285.35	325.63
	b	14.086	16.567	20.006	21.751	22.993	24.027	25.322
Hood	a	93.351	117.38	163	194.75	235.56	269.71	309.25
	b	13.654	15.308	18.65	20.281	21.438	22.299	23.508
Hunt	a	107.65	131.48	178.92	209.36	249.71	282.05	318.9
	b	15.348	16.855	20.456	21.855	22.995	23.713	24.744
Johnson	a	94.751	120.21	168.39	198.98	240.45	275.19	313.38
	b	13.414	15.543	19.272	20.676	21.847	22.804	23.875
Kaufman	a	104.54	132.07	183.2	216.62	260.03	295.03	334.63
	b	14.637	16.912	20.837	22.424	23.65	24.42	25.496
Navarro	a	108.66	132.42	185.55	221.63	268.93	306.83	350.06
	b	15.326	16.758	20.945	22.903	24.437	25.402	26.665
Palo Pinto	a	91.031	115.97	164.22	196.59	242.51	281.03	326.0
	b	13.127	15.264	19.05	20.714	22.468	23.769	25.388
Parker	a	95.164	118.64	166.17	198.53	242.46	279.34	321.89
	b	13.848	15.396	18.999	20.608	22.048	23.123	24.527
Rockwall	a	107.9	131.23	179.89	212.63	254.36	287.68	325.96
	b	15.671	16.882	20.467	22.064	23.178	23.891	24.906
Somervell	a	92.245	116.25	162.12	193.36	232.22	265.8	303.15
	b	13.091	14.967	18.503	20.102	21.066	22.001	23.039
Tarrant	a	95.835	121.96	170.81	203.93	247.1	282.6	322.07
	b	13.425	15.704	19.435	21.09	22.366	23.302	24.388
Wise	a	93.326	118.05	165.95	200.22	247.21	287.89	334.11
	b	13.491	15.315	18.974	20.889	22.662	24.112	25.784

2.1.7.3 Example Problem

A 5-acre site is to be developed in Dallas. Based on site and local information, it is determined that streambank protection is not required and that limiting the 25-year and 100-year storm is also not required. The local government has determined that the development must detain the 2-year and 10-year storms. Rainfall values are taken from Appendix A. The following key information is obtained:

Area = 5 acres
Slope is about 5%
Pre-development t_c = 21 minutes and C factor = 0.22
Post-development t_c = 10 minutes and C factor = 0.80

Steps	2 - year	10 - year
t_c (min)	21	21
i (in/hr)	3.35	4.79
Q_a (equation 2.1.25) (cfs)	3.69	5.27
a (from Table 2.1.7-1)	128.85	210.73
b (from Table 2.1.7-1)	16.624	21.785
V_{pre} (equation 2.1.28a) (ft^3)	16,570	26,042
P_{180} (in)	2.28	3.60
T_d (equation 2.1.27) (min)	51.52	61.69
P_{td} (in)	1.65	2.66
V_{max} (equation 2.1.28b) (ft^3)	22,897	35,245

2.1.8 USGS and TxDOT Regression Methods

2.1.8.1 Introduction

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed.

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas. For this manual, the USGS regression equations are used to determine peak flows in urban drainage areas, and the TXDOT regression equations are used to determine peak flows in rural drainage areas. It may be difficult to choose the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies. Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

The following suggestions should be considered when using regression equations:

- Conduct a field visit to compare and assess the watershed characteristics for comparison with other watersheds.
- Collect all available historical flood data.
- Use the gathered data to interpret any discharge values.

2.1.8.2 USGS Equations for Urban Basins

Regression equations developed by the USGS for urban streams in Dallas-Fort Worth are for estimating peak discharges (QT) having recurrence intervals (T) that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (DA), in square miles, and an urbanization index (UI), which is evaluated as described in the report by Land and others (U.S.G.S., 1982).

The urbanization index is an attempt to more accurately quantify the degree of urbanization by incorporating the factors of storm sewers, curbs and gutters, and channel rectifications. The index is developed by considering these alterations in the upper, middle, and lower third of the drainage basin. Values are assigned to each factor in each one-third of the basin on the basis of the percentage of the subbasin containing that factor. Each factor carries an equal weight regardless of location within the subbasin. The values of each factor vary from 1 to 4, based on the degree of development. The sum of the 9 factors can vary from 9 to 36 and is the value of the urbanization index. The factor values and corresponding percentages of the subbasin affected are listed below:

Percent	Value
0 – 24	1
25 – 49	2
50 – 74	3
75 – 100	4

The following example is given to illustrate the determination of the urbanization index.

Urbanization Index				
	Factors			
Sub area	Storm Sewers	Curbs and Gutters	Channel Rectifications	Total
Upper	4	4	2	10
Middle	3	4	1	8
Lower	3	4	1	8
Urbanization Index				**26**

Source: Techniques for Estimating the Magnitude and Frequency of Floods in the Dallas-Fort Worth Metropolitan Area, Texas, U.S. Geological Survey, Water Resources Investigation 82-18

2.1.8.3 Application of USGS Equations

The USGS regression equations were developed from peak-discharge records from drainage areas in the Dallas-Fort Worth area ranging from 1.25 to 66.4 square miles with results considered applicable to drainage areas between 3 and 40 square miles having urbanization indexes between 12 and 33. The standard errors of estimate of the regression equations are about 30 percent. As such, the USGS regression method should only be used for calculating peak discharge in urban drainage areas as described.

The USGS method can be used for several design applications, including storm drain systems, culverts, small drainage ditches and open channels, and energy dissipaters.

For a complete description of the USGS regression equations presented below, consult the USGS publication *Techniques for estimating the magnitude and frequency of floods in the Dallas-Fort Worth metropolitan area, Texas: U.S. Geological Survey Water-Resources Investigations Report 82-18, 55 p*. Table 2.1.8-1 gives the USGS regression equations for urban streams in the Dallas-Fort Worth area.

2.1.8.4 Peak Discharge Limitations for Urban Basins

Following are the limitations of the variables within the peak discharge equations. These equations should not be used on drainage areas which have physical characteristics outside the limits listed below:

Physical Characteristics	Minimum	Maximum	Units
A - Drainage Area	3	40	mi^2
UI – Urbanization Index	12	33	

Table 2.1.8-1 USGS Peak Flow Regression Equations for Dallas-Fort Worth Urban Areas

Frequency	Equations
2-year	$Q_2 = 42.83(DA)^{0.704}(UI)^{0.836}$
5-year	$Q_5 = 82.92(DA)^{0.724}(UI)^{0.751}$
10-year	$Q_{10} = 120.7(DA)^{0.735}(UI)^{0.697}$
25-year	$Q_{25} = 184.8(DA)^{0.745}(UI)^{0.632}$
50-year	$Q_{50} = 246.4(DA)^{0.752}(UI)^{0.587}$
100-year	$Q_{100} = 362.1(DA)^{0.752}(UI)^{0.510}$

For these equations: DA = drainage area in mi^2, UI = urbanization index

Source: USGS, 1982

2.1.8.5 TxDOT Equations for Rural (or Undeveloped) Basins

The Texas Department of Transportation (TxDOT) has a regression method for estimating peak discharges for rural basins. For a complete discussion of the development of these equations consult Chapter 5, Section 11 of the TxDOT Hydraulic Design Manual, available online at http://manuals.dot.state.tx.us/docs/colbridg/forms/hyd.pdf or the reference USGS, 1997.

2.1.8.6 Rural (or Undeveloped) Basin Application

Equation 2.1.29 applies to rural, uncontrolled watersheds. Figure 2.1.8-1 presents the geographic extents of each region. Note that most of the NCTCOG region lies within Region 7, with small portions of Region 3 and 4. Table 2.1.8-2 presents the coefficients and limits of applicability for Regions 3, 4, and 7. Generally, use this equation to compare with the results of other methods, check existing structures, or where it is not practicable to use any other method, keeping in mind the importance of the facility being designed.

$$Q_T = aA^bSH^cSL^d \qquad (2.1.29)$$

where:

Q_T = T-year discharge (cfs)

A = contributing drainage area (sq. mi.)

SH = basin-shape factor defined as the ratio of main channel length squared to contributing drainage area (sq. mi./sq. mi.)

SL = mean channel slope defined as the ratio of headwater elevation of longest channel minus main channel elevation at site to main channel length (ft./mi.). Note: This differs from previous rural regression equations in which slope was defined between points 10 and 85 percent of the distance along the main channel from the outfall to the basin divide.

a, b, c, d = multiple linear regression coefficients dependent on region number and frequency.

The equations to be used for Regions 3, 4, and 7 are found in Table 2.1.8-2.

Regions 3, 4, and 7 have two sets of coefficients. For these regions, if the drainage area is between 10 and 100 sq. mi., determine a weighted discharge (Q_w) as shown in Equation 2.1.30.

$$Q_w = (2 - \log(A/z))Q_1 + (\log(A/z)-1)Q_2 \qquad (2.1.30)$$

where:

Q_w = weighted discharge (cfs)

A = contributing drainage area (sq. mi.)

z = 1.0 for English measurements units

Q_1 = discharge based on regression coefficients for A < 32 sq. mi. (cfs)

Q_2 = discharge based on regression coefficients for A ≥ 32 sq. mi. (cfs)

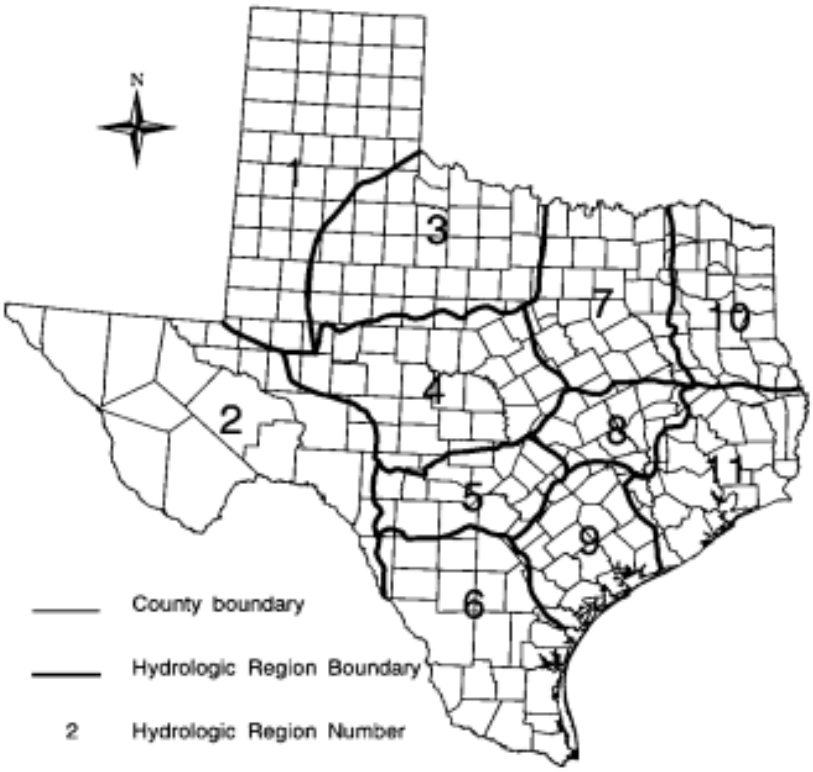


Figure 2.1.8-1 Hydrologic Regions for Statewide Rural Regression Equations
Source: TXDOT, 2002

Table 2.1.8-2 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas[1]

[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]

Hydrologic region and recurrence interval	Weighted least-squares regression equation for corresponding recurrence interval	Range of indicated independent variables in corresponding region (units as noted)
Region 3 (sites with contributing drainage area less than 32 square miles)[2]		
2 yr	$Q_2 = 119\ A^{.592}$	A: 0.10 to 97.0
5 yr	$Q_5 = 252\ A^{.629}$	
10 yr	$Q_{10} = 373\ A^{.652}$	SH: 0.16 to 9.32
25 yr	$Q_{25} = 566\ A^{.679}$	
50 yr	$Q_{50} = 743\ A^{.698}$	SL: 10.7 to 105
100 yr	$Q_{100} = 948\ A^{.715}$	
Region 3 (sites with contributing drainage area greater than 32 square miles)[2]		
2 yr	$Q_2 = 8.05\ A^{.668}\ SL^{.659}\ SH^{.189}$	A: 11.8 to 14,635
5 yr	$Q_5 = 4.20\ A^{.626}\ SL^{.574}$	
10 yr	$Q_{10} = 91.9\ A^{.579}\ SL^{.537}$	SH: 1.71 to 75.0
25 yr	$Q_{25} = 233\ A^{.523}\ SL^{.476}$	
50 yr	$Q_{50} = 448\ A^{.484}\ SL^{.425}$	SL: 4.81 to 36.3
100 yr	$Q_{100} = 835\ A^{.447}\ SL^{.372}$	
Region 4 (sites with contributing drainage area less than 32 square miles)[2]		
2 yr	$Q_2 = 97.1\ A^{.626}$	A: 0.19 to 81.1
5 yr	$Q_5 = 196\ A^{.650}\ SH^{.257}$	
10 yr	$Q_{10} = 293\ A^{.697}\ SH^{.281}$	SH: 0.05 to 6.52
25 yr	$Q_{25} = 455\ A^{.741}\ SH^{.311}$	
50 yr	$Q_{50} = 53\ A^{.927}\ SL^{.558}\ SH^{.333}$	SL: 13.5 to 226
100 yr	$Q_{100} = 51\ A^{.968}\ SL^{.627}\ SH^{.353}$	
Region 4 (sites with contributing drainage area greater than 32 square miles)[2]		
2 yr	$Q_2 = 0.00660\ A^{1.29}\ SL^{2.09}$	A: 12 to 19,819
5 yr	$Q_5 = 0.0212\ A^{1.24}\ SL^{2.18}$	
10 yr	$Q_{10} = 0.0467\ A^{1.20}\ SL^{2.18}$	SH: 0.49 to 19.7
25 yr	$Q_{25} = 0.102\ A^{1.16}\ SL^{2.18}$	
50 yr	$Q_{50} = 0.166\ A^{1.13}\ SL^{2.19}$	SL: 3.52 to 36.1
100 yr	$Q_{100} = 0.252\ A^{1.11}\ SL^{2.19}$	
Region 7 (sites with contributing drainage area less than 32 square miles)[2]		
2 yr	$Q_2 = 832\ A^{.568}\ SL^{-.285}$	A: 0.20 to 78.7
5 yr	$Q_5 = 584\ A^{.610}$	
10 yr	$Q_{10} = 831\ A^{.592}$	SH: 0.037 to 36.6
25 yr	$Q_{25} = 1196\ A^{.576}$	
50 yr	$Q_{50} = 1505\ A^{.566}$	SL: 7.25 to 116
100 yr	$Q_{100} = 1842\ A^{.558}$	

Table 2.1.8-2 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas[1]

[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]

Hydrologic region and recurrence interval	Weighted least-squares regression equation for corresponding recurrence interval	Range of indicated independent variables in corresponding region (units as noted)
Region 7 (sites with contributing drainage area greater than 32 square miles)[2]		
2 yr	$Q_2 = 129\ A^{.578}\ SL^{.364}$	A: 13 to 2,615
5 yr	$Q_5 = 133\ A^{.605}\ SL^{.578}$	
10 yr	$Q_{10} = 178\ A^{.644}\ SL^{.699}\ SH^{-.239}$	SH: 1.66 to 36.6
25 yr	$Q_{25} = 219\ A^{.651}\ SL^{.776}\ SH^{-.267}$	
50 yr	$Q_{50} = 261\ A^{.653}\ SL^{.817}\ SH^{-.291}$	SL: 3.85 to 31.9
100 yr	$Q_{100} = 313\ A^{.654}\ SL^{.849}\ SH^{-.316}$	

1. Source: U.S.G.S., 1997, pp. 62-65.
2. Use Equation 2.1.29 to calculate a weighted discharge for streams with contributing drainage area falling within the arrange of 10 to 100 square miles.

2.1.8.7 Example Problem

For the 100-year flood, calculate the peak discharge for a rural drainage area located in Region 7 on Timber Creek near Collinsville, Texas.

- Drainage Area = 38.8 mi^2
- Main Channel Slope = 13.13 ft/mi
- Main Channel Length= 14.24 mi.
- Shape Factor = (channel miles)^2 divided by Area = 5.23

<u>Peak Discharge Calculations</u>

The 100-year Rural Peak Discharge determination will necessitate the use of Equations 2.1.29 and 2.1.30 because the drainage is in the range of 10-100.

$Q_1 = 1842\ A^{.558} = 1842(38.8)^{.558} = 14{,}186$ cfs

$Q_2 = 313\ A^{.654}\ SL^{.849}\ SH^{-.316}$
$= 313(38.8)^{.654}\ (13.13)^{.849}\ (5.23)^{-.316}$
$= 18{,}072$ cfs

$Q_{100} = (2\text{-}\log(A))Q_1 + (\log(A)\text{-}1)Q_2$
$= (2 - \log(38.8))14{,}186 + (\log(38.8)\text{-}1)\ 18{,}072$ cfs
$= 16{,}474$ cfs

2.1.9 Downstream Hydrologic Assessment

The purpose of the Streambank Protection and Flood Control criteria is to protect downstream properties from flood and erosion impacts due to upstream development. Some Local Criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff.

Due to a site's location within a watershed, there may be very little reason for requiring flood control from a particular site. In certain circumstances where detention is in place or a master drainage plan has been adopted, a development may receive or plan to receive less that ultimate developed flow conditions from upstream. This might be considered in the detention needed and its influence on the downstream assessment. Any consideration in such an event would be with the approval of the local authority. This section outlines a suggested procedure for determining the impacts of post-development storm water peak flows and volumes that a community may require as part of a developer's storm water management site plan.

2.1.9.1 Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 2.1.9-1. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required.

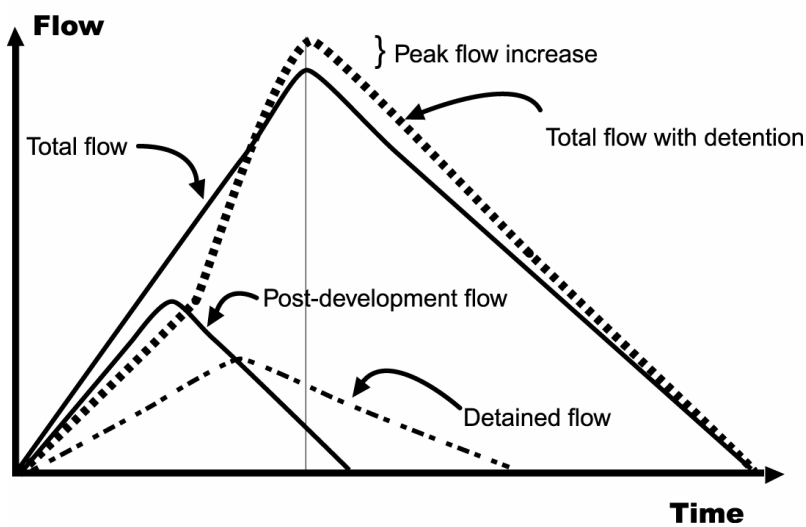


Figure 2.1.9-1 Detention Timing Example

In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained. This is most likely to happen if detention is placed on tributaries towards the bottom of the watershed, holding back peak flows and adding them as the peak from the upper reaches of the watershed arrives.

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

Figure 2.1.9-2 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

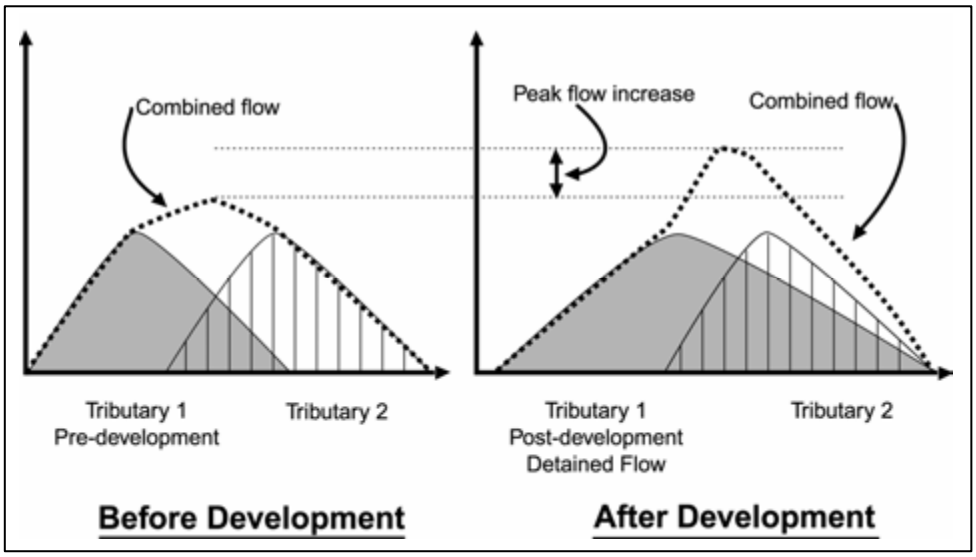


Figure 2.1.9-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

2.1.9.2 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on storm water peak flows and velocities are evaluated downstream. The assessment extends from an outfall of a development to a point downstream, determined by one of two methods:

- *Zone of Influence* – Point downstream where the discharge from a proposed development no longer has a significant impact upon the receiving stream or storm drainage system
- *Adequate Outfall* – Location of acceptable outfall that does not create adverse flooding or erosion conditions downstream

These methods recognize the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be felt. Beyond this zone of influence the storm water effects of a structural control become relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, a general rule of thumb is that the zone of influence can be considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. This is known as the *10% Rule*. As an example, if a structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in a downstream assessment include:

1. Determine the outfall location of the site and the pre- and post-development site conditions.
2. Using a topographic map determine a preliminary lower limit of the zone of influence (approximately 10% point).
3. Using a hydrologic model determine the pre-development peak flows and velocities at each junction beginning at the development outfall and ending at the next junction beyond the 10% point. Undeveloped off-site areas are modeled as "full build-out" for both the pre- and post-development analyses. The discharges and velocities are evaluated for three storms:

   "Streambank Protection" storm, either the 1- or 2- year, 24-hour event

   "Conveyance" storm of either the 5-, 10-, or 25-year, 24-hour event

   100-year, 24-hour storm event
4. Change the land use on the site to post-development conditions and rerun the model.
5. Compare the pre- and post-development peak discharges and velocities at the downstream end of the model. If the post-developed flows are higher than the pre-developed flows for the same frequency event, or the post-developed velocities are higher than the allowable velocity of the downstream receiving system, extend the model downstream. Repeat steps 3 and 4 until the post-development flows are less than the pre-developed flows, and the post-developed velocities are below the allowable velocity. Allowable velocities are given in Table 4.4-3 in Chapter 4 of this manual.
6. If shown that no peak flow increases occur downstream, and post-developed velocities are allowable, then the control of the flood protection volume (Q_f) can be waived by the local authority. The developer saves the cost of sizing a detention basin for flood control. In this case the downstream assessment saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
7. If peak discharges are increased due to development, or if downstream velocities are erosive, one of the following options are required.
   - Document that existing downstream conveyance is adequate to convey post-developed storm water discharges (Option 1 for Streambank Protection and Flood Control)
   - Work with the local government to reduce the flow elevation and/or velocity through channel or flow conveyance structure improvements downstream. (Option 2 for Streambank Protection and Flood Control)
   - Design an on-site structural control facility such that the post-development flows do not increase the peak flows, and the velocities are not erosive, at the outlet and the determined junction locations.

Even if the results of the downstream assessment indicate that no downstream flood or erosion protection is required, the water quality steps of the *integrated* Design Approach will still need to be addressed.

2.1.9.3 Example Problem

Figure 2.1.9-3 illustrates the concept of the ten-percent rule for two sites in a watershed.

Discussion

Site A is a development of 10 acres, all draining to a wet Extended Detention (ED) storm water pond. The flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase or erosive velocities at the 80-acre point then the same will be true through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single, "full build-out" condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase, and velocities are not erosive, at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows a detention facility, in this case, will actually increase the peak flow in the stream.

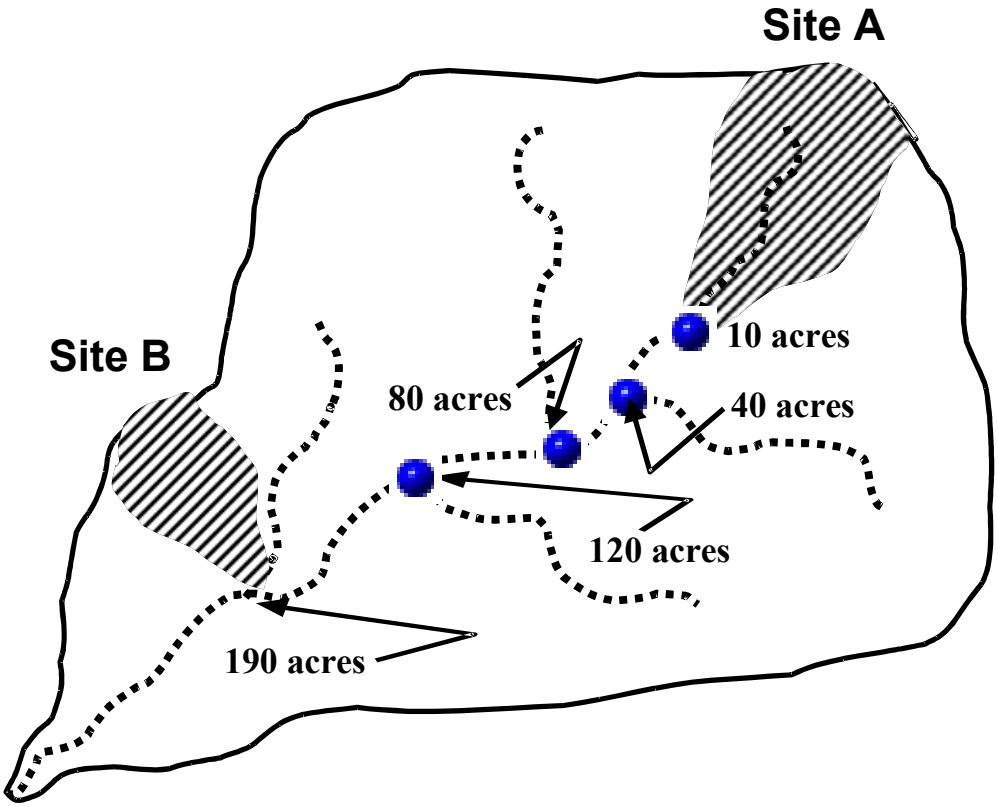


Figure 2.1.9-3 Example of the Ten-Percent Rule

2.1.10 Water Quality Protection Volume and Peak Flow

2.1.10.1 Water Quality Protection Volume Calculation

The Water Quality Protection Volume (WQ_v) is the treatment volume required to remove a significant percentage of the storm water pollution load, defined in this Manual as a 70 - 80% removal of the average annual post-development total suspended solids (TSS) load. This is achieved by intercepting and

treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year. The WQ_v should not be discharged in a period less than 24 hours. Detention times greater than 24 hours would be more effective. Procedures for computing the release rate are found in Section 4.6.3.

The water quality protection volume is calculated by multiplying the 85th percentile annual rainfall event by the volumetric runoff coefficient (R_v) and the site area. R_v is defined as:

$$R_v = 0.05 + 0.009(I) \qquad (2.1.31)$$

where:

I = percent of impervious cover (%)

For North Central Texas, the average 85th percentile annual rainfall event is 1.5 inches. Therefore, WQ_v is calculated using the following formula:

$$WQ_v = \frac{1.5\ R_v\ A}{12} \qquad (2.1.32)$$

where:

WQ_v = water quality protection volume (acre-feet)

R_v = volumetric runoff coefficient

A = total drainage area (acres)

WQ_v can be expressed in inches ($\mathbf{Q_{wv}}$) using the following formula:

$$Q_{wv} = 1.5(R_v) \qquad (2.1.33)$$

where:

$\mathbf{Q_{wv}}$ = water quality protection volume (inches)

2.1.10.2 Water Quality Protection Volume Peak Flow Calculation

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as for sand filters and infiltration trenches. An arbitrary storm would need to be chosen using the Rational Method, as conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the Water Quality Protection Volume and the simplified peak flow estimating method above. A brief description of the calculation procedure is presented below.

(Step 1) Using Q_{wv} and equation 2.1.8, a corresponding Curve Number (CN) is computed utilizing the following equation:

$$CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\ Q_{wv}P)^{1/2}]$$

where:

P = rainfall, in inches (use 1.5 inches for the iSWM Water Quality Storm)

Q_{wv} = Water Quality Protection Volume, in inches ($1.5R_v$)

(Step 2) Once a CN is computed, the time of concentration (t_c) is computed (based on the methods described in this section).

(Step 3) Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the water quality storm event is computed using a slight modification of the Simplified SCS Peak Runoff Rate Estimation technique of subsection 2.1.5.7. Use Type II rainfall distribution for North Central Texas.

(Step 4) Read initial abstraction (I_a) from Table 2.1.5-3, compute I_a/P

(Step 5) Read the unit peak discharge (q_u) from Figure 2.1.5-6 for appropriate tc

- Using Q_{wv}, compute the peak discharge (Q_{wq})

$$\mathbf{Q_{wq} = q_u * A * Q_{wv}}$$

where:

Q_{wq} = the water quality peak discharge (cfs)
q_u = the unit peak discharge (cfs/mi²/inch)
A = drainage area (mi^2)
Q_{wv} = Water Quality Protection Volume, in inches ($1.5R_v$)

2.1.10.3 Example Problem

Using the data and information from the example problem in subsection 2.1.5.8 calculate the water quality volume and the water quality peak flow.

Calculate water quality protection volume (Q_{wv})

Compute volumetric runoff coefficient, R_v

$R_V = 0.05 + (0.009)(I) = 0.05 + (0.009)(18/50 \times 100\%) = 0.37$

Compute water quality protection volume, Q_{wv}

$Q_{wv} = 1.5(R_V)(A)/12 = 1.5(.37)(50)/12 = 2.31$ acre-feet

Calculate water quality peak flow

Compute runoff volume in inches, Q_{wv}, where P = 1.5 inches:

$Q_{wv} = 1.5\ R_v = 1.5 * 0.37 = 0.56$ inches

Compute curve number:

$CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\ Q_{wv}\ P)^{1/2}]$
$CN = 1000/[10 + 5*1.5 + 10*0.56 - 10(0.56^2 + 1.25*0.56*1.5)^{1/2}]$
$= 88$

$t_c = 0.35$ (computed previously)

$S = 1000/CN – 10 = 1000/84 – 10 = 1.36$ inches
$0.2S = I_a = 0.27$ inches
$I_a/P = 0.27/1.5 = 0.18$

Find q_u:

From Figure 2.1.5-6 for $I_a/P = 0.18$ $q_u = 580$ cfs/mi^2/in

Compute water quality peak flow:

$Q_{wq} = q_u * A * Q_{wv} = 580 * 50/640 * 0.56 = 25.4$ cfs

2.1.11 Streambank Protection Volume Estimation

2.1.11.1 Streambank Protection Volume Calculation

The Simplified SCS Peak Runoff Rate Estimation approach (see subsection 2.1.5.7) can be used for estimation of the Streambank Protection Volume (SP_v) for storage facility design.

This method should not be used for standard detention design calculations. See the modified rational method in Section 2.1.7 for preliminary detention calculations without formal routing or the SCS Hydrologic Method in Section 2.1.5.

For SP_v estimation, using Figure 2.1.5-6 in Section 2.1.5, the unit peak discharge (q_U) can be determined based on I_a/P and time of concentration (t_C). Knowing q_U and T (extended detention time, typically 24 hours), the q_O/q_I ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 2.1.11-1.

Using the following equation from TR-55 for a Type II rainfall distribution, V_S/V_r can be calculated.

Note: Figure 2.1.11-2 can also be used to estimate V_S/V_r.

$$V_S/V_r = 0.682 - 1.43\,(q_O/q_I) + 1.64\,(q_O/q_I)^2 - 0.804\,(q_O/q_I)^3 \qquad (2.1.34)$$

where:

V_S = required storage volume (acre-feet)
V_r = runoff volume (acre-feet)
q_O = peak outflow discharge (cfs)
q_I = peak inflow discharge (cfs)

The required storage volume can then be calculated by:

$$V_S = \frac{(V_S/V_r)(Q_d)(A)}{12} \qquad (2.1.35)$$

where:

V_S and V_r are defined above
Q_d = the developed runoff for the design storm (inches)
A = total drainage area (acres)

While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 25-year storm.

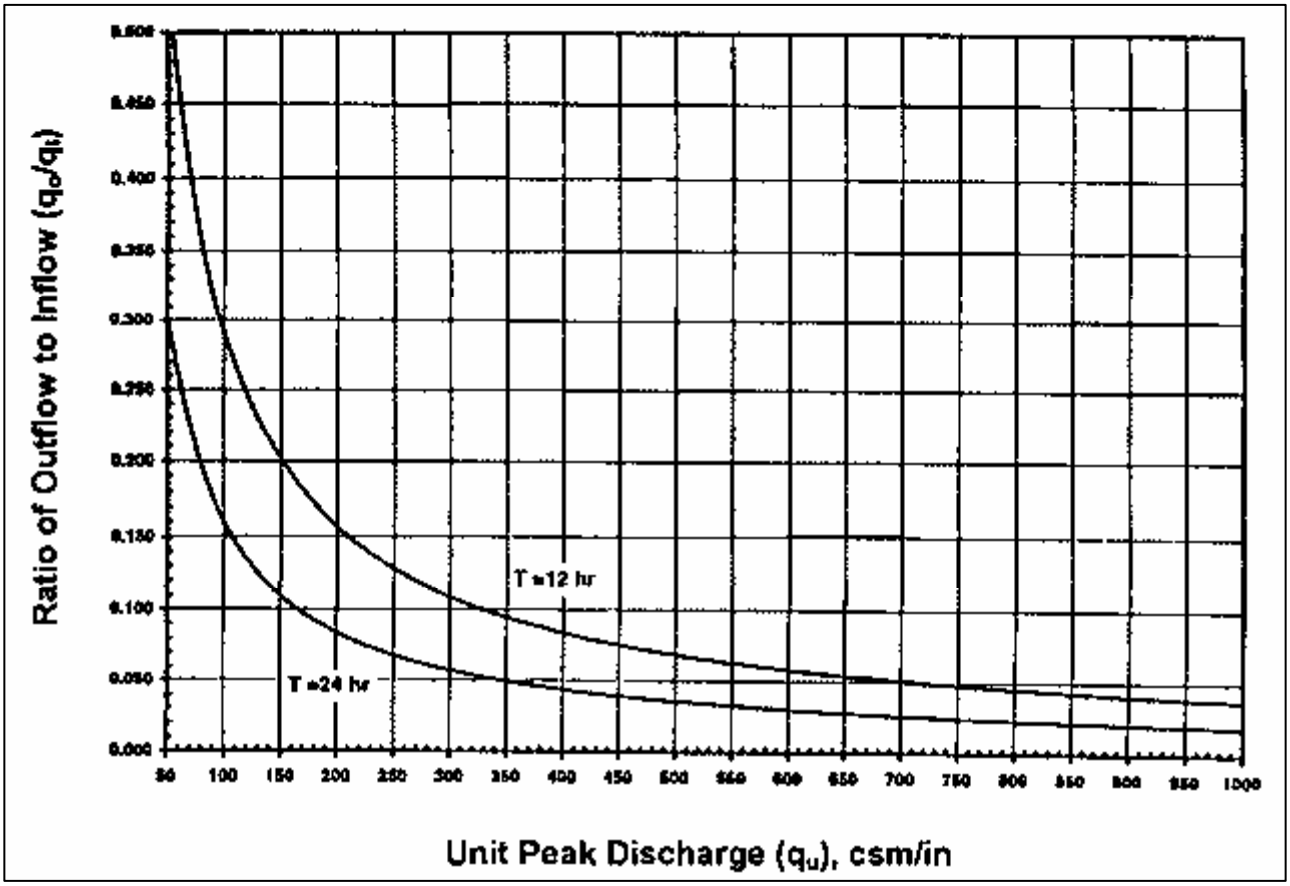


Figure 2.1.11-1 Detention Time vs. Discharge Ratios
(Source: MDE, 1998)

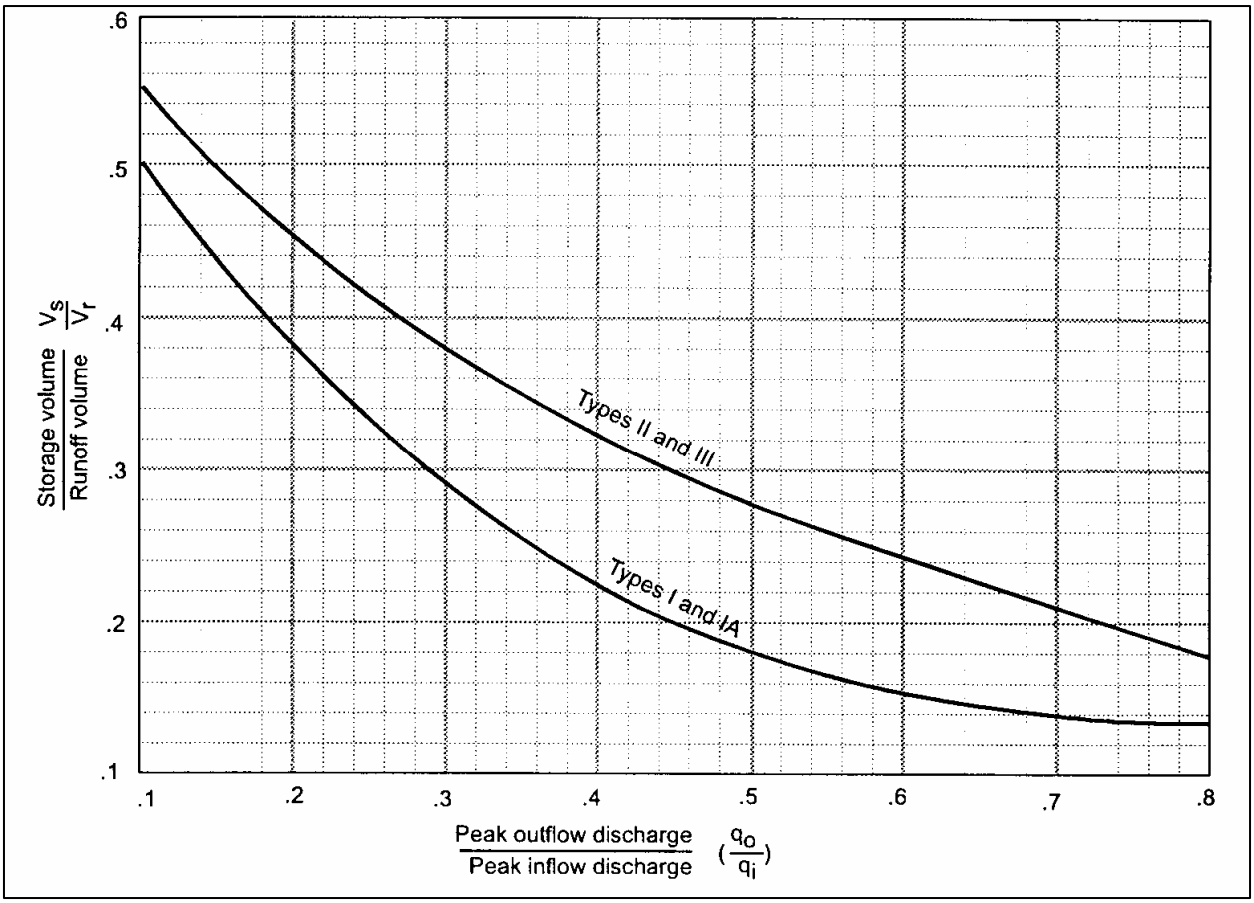


Figure 2.1.11-2 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
(Source: TR-55, 1986)

2.1.11.2 Example Problem

Compute the Streambank Protection Volume (SP_v) for the 50-acre watershed in example 2.1.5.8.

Computations

8. Calculate rainfall excess:
   - The 1-year, 24 hour rainfall is 2.64 inches (0.11 in/hr x 24 hours – From Appendix A, Table A-16).
   - Composite area-weighted Curve Number is 83.
   - From equation 2.1.7, Q_d (1-year developed) = 1.2 inches

9. Calculate time of concentration

   t_c = 20.86 minutes (.35 hours)

10. Calculate I_a/P for CN = 83; I_a = .410 (Table 2.1.5-3)

    I_a/P = (.410 / 2.64) = .155 (Note: Use straight-line interpolation to facilitate use of Figure 2.1.5-6.)

11. Find unit discharge q_u:
    From Figure 2.1.5-6 for I_a/P = .155 and t_c = .35 hr
    q_u = 600 csm/in

12. Find discharge ratio q_O/q_I:

    From Figure 2.1.11-1 for q_u = 600 csm/in and T = 24 hr
    $q_O/q_I = 0.03$

13. Calculate streambank protection volume ($SP_v = V_S$)

    For a Type II rainfall distribution,
    $V_S/V_r = 0.682 – 1.43 (q_O/q_I) + 1.64 (q_O/q_I)^2 – 0.804 (q_O/q_I)^3$
    $V_S/V_r = 0.682 – 1.43 (0.03) + 1.64 (0.03) – 0.804 (0.03) = 0.64$

Therefore, streambank protection volume with Q_d (1-year developed) = 1.2 inches, from Step 1, is

$SP_v = V_S = (V_S/V_r)(Q_d)(A)/12 = (0.64)(1.2)(50)/12 = 3.20$ acre-feet

2.1.12 Water Balance Calculations

2.1.12.1 Introduction

Water balance calculations can help determine if a drainage area is large enough, or has the right characteristics, to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for retention pond and wetland design.

The details of a rigorous water balance are beyond the scope of this manual. However, a simplified procedure is described herein to provide an estimate of pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

2.1.12.2 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential):

$$\Delta V = \Sigma I - \Sigma O \qquad (2.1.36)$$

where:

- Δ = "change in"
- V = pond volume (ac-ft)
- Σ = "sum of"
- I = Inflows (ac-ft)
- O = Outflows (ac-ft)

The inflows consist of rainfall, runoff, and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 2.1.36 can be changed to reflect these factors.

$$\Delta V = P + R_o + B_f – I – E – E_t – O_f \qquad (2.1.37)$$

where:

- V = volume (ac-ft)
- P = precipitation (ac-ft) = (Rainfall in Inches times area in acres divided by 12)
- R_o = runoff (ac-ft)
- B_f = baseflow (ac-ft)
- I = infiltration (ac-ft) (Use Equation 2.1.33)

E = evaporation (ac-ft) (Surface evaporation in feet times surface area)
E_t = evapotranspiration (ac-ft)
O_f = overflow (ac-ft)
Δ = "change in" (+ gain; - loss)

Rainfall (P) – Monthly rainfall values can be obtained from National Weather Service climatology data at:

http://www.srh.noaa.gov/fwd/ntexclima.html

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) and divided by 12, it becomes acre-feet of volume. Table 2.1.12-1 shows monthly rainfall rates for the Dallas-Fort Worth area based on a 30-year period of record at Dallas-Fort Worth International Airport.

Runoff (R_o) – Runoff is equivalent to the rainfall for the period times the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall. In lieu of gage information, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model. Two other methods have been proposed.

Equation 2.1.31 gives the volumetric coefficient (R_v) of the drainage area. If it can be assumed that the average storm producing runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Texas, this is equivalent of about a 10% runoff volume loss. Thus a factor of 0.9 should be applied to the calculated R_v value to account for storms producing no runoff. Equation 2.1.38 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$R_o = 0.9(P/12)R_vA \qquad (2.1.38)$$

where:

P = precipitation (in)
R_o = runoff volume (acre-ft)
R_v = volumetric runoff coefficient [see equation 2.1.31]
A = Area in acres

Table 2.1.12-1 Monthly Precipitation Values

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation (in)	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57
Annual Precipitation (in) 34.73												

Source: National Weather Service, 2002

Baseflow (B_f) – Most storm water ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks. Consideration may also have to be given to irrigation return flow in certain areas.

Infiltration (I) – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation as:

$$I = Ak_hG_h \qquad (2.1.39)$$

where:

I = infiltration (ac-ft/day)

A = cross sectional area through which the water infiltrates (ac)

K_h = saturated hydraulic conductivity or infiltration rate (ft/day)

G_h = hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate Table 2.1.12-2 can be used.

Table 2.1.12-2 Saturated Hydraulic Conductivity

Material	Hydraulic Conductivity	
	in/hr	ft/day
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08
Clay	0.02	0.04

Source: Ferguson and Debo, "On-Site Storm Water Management," 1990

Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used. A pan coefficient of 0.7 is commonly used to convert the higher pan value to the lower lake values.

Table 2.1.12-3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information for Grapevine, Texas. Figure 2.1.12-1 depicts a map of annual free water surface (FWS) evaporation averages for Texas based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural storm water ponds and wetlands being designed in Texas. Total annual values can be estimated from this map and distributed according to Table 2.1.12-3.

Table 2.1.12-3 Evaporation Monthly Distribution – Grapevine, Texas as a % of Annual Total

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
3.1%	4.0%	7.2%	8.7%	10.3%	12.4%	14.5%	13.9%	9.8%	7.4%	4.9%	3.9%

Evapotranspiration (E_t). Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of E_t for crops in Texas is well documented and has become standard practice. However, for wetlands the estimating methods are not documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating E_t only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based E_t estimates and a decision made. Crop-based E_t estimates can be obtained from typical hydrology textbooks or from the web site mentioned above.

Overflow (O_f) – Overflow is considered as excess runoff, and in water balance design is either not considered, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in pond design.

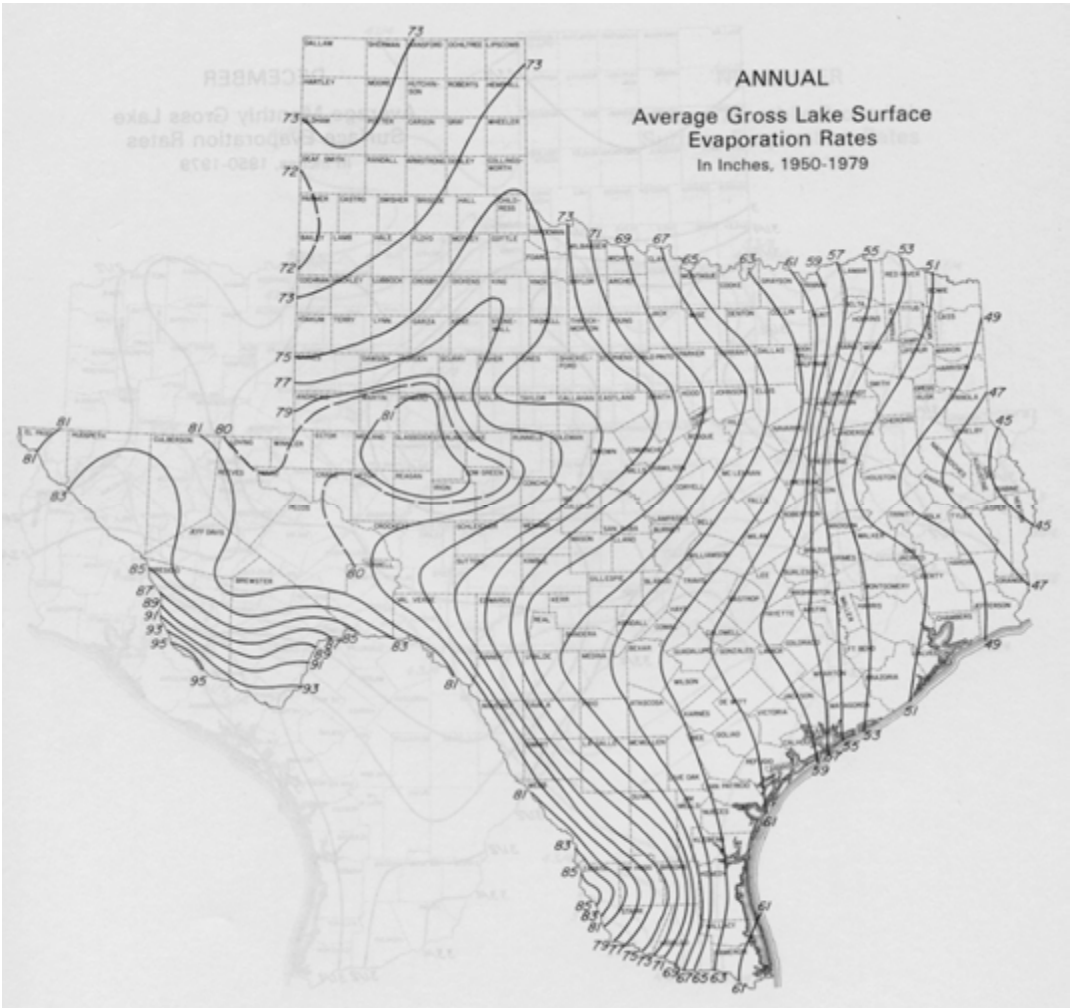


Figure 2.1.12-1 Average Annual Free Water Surface Evaporation (in inches)
(Source: NOAA, 1982)

2.1.12.3 Example Problem

A 26-acre site in North Dallas is being developed along with an estimated 0.5-acre surface area pond. There is no baseflow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data, we find the site is 75% impervious with clay loam soil.

- From equation 2.1.31, R_v = 0.05 + 0.009 (75) = 0.73. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 2.1.12-1 is about 64 inches.
- For a clay loam the infiltration rate is I = 0.18 ft/day (Table 2.1.12-2).
- From a grading plan, it is known that about 10% of the total pond area is sloped greater than 1:4.
- Monthly rainfall for Dallas was found from a Web site similar to the one provided above.

Table 2.1.12-4 shows summary calculations for this site for each month of the year.

Table 2.1.12-4 Summary Information for the North Dallas Site

Drainage Area (Acres)	26
Pond Surface (Acres)	0.5
Volume at Overflow (Ac-Ft)	2
Watershed Efficiency	0.65
Annual Rainfall	34.73
Infiltration Rate (In/Day)	0.18 Clay Loam
% Pond Bottom Flat (Acres)	90
% Pond Bottom > 1:4 (Acres)	10
Annual Lake Evaporation (in)	64 Assume Pond Starts Full

1	Months of Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2	Days Per Month	31	28	31	30	31	30	31	31	30	31	30	31
3	Monthly Precipitation	1.9	2.37	3.06	3.2	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57
4	Evaporation - % of Yr	3.1	4	7.2	8.7	10.3	12.4	14.5	13.9	9.8	7.4	4.9	3.9
5	Runoff (Ac-Ft)	2.68	3.34	4.31	4.51	7.25	4.55	2.99	2.86	3.41	5.79	3.62	3.62
6	Precipitation (Ac-Ft)	0.08	0.10	0.13	0.13	0.21	0.13	0.09	0.08	0.10	0.17	0.11	0.11
7	Evaporation (Ac-Ft)	0.08	0.11	0.19	0.23	0.27	0.33	0.39	0.37	0.26	0.20	0.13	0.10
8	Infiltration (Ac-Ft)	2.65	2.39	2.65	2.57	2.65	2.57	2.65	2.65	2.57	2.65	2.57	2.65
9													
10	Balance (Ac-Ft)	0.02	0.94	1.59	1.84	4.54	1.79	0.04	-0.08	0.68	3.11	1.03	0.97
11	Running Balance (Ac-Ft)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.92	2.00	2.00	2.00	2.00

Explanation of Table line number:

1. Months of year
2. Days per month in percent
3. Monthly precipitation
4. Distribution of evaporation by month
5. In the example, watershed efficiency of 0.65 times the rainfall and area (in acres) and converted to acre-feet. The Watershed efficiency must be determined for each watershed.
6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet

7. Evaporation equals the monthly percentage of the annual gross lake evaporation in inches converted to acre-feet
8. Infiltration equals infiltration rate times 90% of the surface area plus infiltration rate times 0.5 (banks greater than 1:4) times 10% of the pond area converted to acre-feet
10. Lines 5 and 6 minus lines 7 and 8
11. Accumulated total from line 10 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design. Each pond has a unique volume at which overflows occur and it would be used for line 11. The pond volume in January should be set equal to the expected end-of-year volume.

It can be seen that, for this example, the pond has potential to maintain a wet pond in all months. Had the soil been a sandy clay loam with an infiltration rate of 0.34 inches per day, the pond would have been dry most months of the year. Excessive infiltration rates may be remedied in a number of ways including compacting the pond bottom, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area.

Climatic data for North Texas, as that in Figure 2.1.12-2, can be obtained from the following web site: http://www.srh.noaa.gov/fwd/CLIMO/dfw/normals/dfwann.html.

DFW Annual Summary of Normal, Means, and Extremes

Rain (in.)

		POR	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	YEAR
Normal		30	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57	34.73
Monthly Maximum		45	5.07	7.40	6.69	12.19	13.66	8.75	11.13	6.85	9.52	14.18	6.23	8.75	14.18
Year of Occurrence			1998	1997	1995	1957	1982	1989	1973	1970	1964	1981	1964	1991	Oct 1981
Minimum Monthly		45	T	0.15	0.10	0.11	0.95	0.40	0	T	0.09	T	0.20	0.17	0
Year of Occurrence			1986	1963	1972	1987	1996	1964	1993	1980	1984	1975	1970	1981	Jul 1993
Max in 24 hours		45	3.15	4.06	4.39	4.55	5.34	3.15	3.76	4.05	4.76	5.91	2.83	422	5.91
Year of Occurrence			1998	1965	1977	1957	1989	1989	1975	1976	1965	1959	1964	1991	Oct 1959
Number of Days with…	Precipitation > Tr.	30	6.7	6.3	7.3	7.6	8.7	6.4	4.7	4.6	7.1	6.2	6.0	6.5	78.1
	Precipitation > 0.99	30	0.3	05	0.7	1.2	1.4	0.9	0.7	0.8	1.1	1.4	0.6	0.4	10.0

Figure 2.1.12-2 Dallas/Fort Worth Precipitation Information

References

Bureau of Reclamation "Flood Hydrology Manual, A Water Resources Technical Publication"

Bedient, Philip B., and Huber, Wayne C., 1988, Hydrology and Floodplain Analysis, Addison–Wesley Publishing Company, Reading, Massachusetts.

Debo, Thomas N., and Andrew J. Reese, 2003. Municipal Storm Water Management. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

Ferguson, B. and Debo, T., 1990. On-Site Storm water Management.

Hershfield, D. M., 1961. Rainfall Frequency Atlas of the United States, Technical Paper 40.

Maryland Department of the Environment, 2000. Maryland Storm water Design Manual, Volumes I and II. Prepared by Center for Watershed Protection (CWP).

Nelson, Thomas L, Fort Worth District, August 1970, Synthetic Unit Hydrograph Relationships, Trinity River Tributaries, Fort Worth–Dallas Urban Area, Hydrologic Engineering Seminar on Urban Hydrology, Sacramento District, Hydrologic Engineering Center (USACE), Davis, California.

NOAA, 1977. Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States, NOAA Technical Memo. NWS HYDRO-35.

NOAA, 1982. Evaporation Atlas for the Contiguous 48 United States, NOAA Technical Report NWS 33.

Pitt, Robert, 1994. Small Storm Hydrology. Unpublished report. Department of Civil Engineering, University of Alabama at Birmingham.

Rodman, Paul K., October 1977, Effects of Urbanization on Various Frequency Peak Discharges, Fall Meeting, Texas, New Mexico, and Mexico Sections, ASCE, Albuquerque, New Mexico.

Schueler, Thomas R., 1987. Controlling Urban Runoff. Washington Metropolitan Water Resources Planning Board.

Texas Department of Water Resources (TDWR), December 1983. Climatic Atlas of Texas. LP-192.

U.S. Army Corps of Engineers, 1959, Flood-Hydrograph Analysis and Computations, Engineering Manual 1110-2-1405.

U.S. Army Corps of Engineers, May 1986, Determination of Percent Sand in Watersheds, Hydrologic Engineering Section, Hydrology and Hydraulics Branch, Fort Worth District, Fort Worth, Texas.

U.S. Army Corps of Engineers, Hydrologic Engineering Center, March 2000, Hydrologic Modeling System Hec-HMS, Technical Reference Manual, Davis, California.

U.S. Army Corps of Engineers, Hydrologic Engineering Center, 2001, Hydrologic Modeling System Hec-HMS, User's Manual, Davis, California.

U. S. Department of Agriculture, Soil Conservation Service, Engineering Division, 1985. SCS National Engineering Handbook.

U. S. Department of Agriculture, Soil Conservation Service, Engineering Division, 1986. Urban Hydrology for Small Watersheds. Technical Release 55 (TR-55).

U. S. Department of Transportation, Federal Highway Administration, 1984. Hydrology. Hydraulic Engineering Circular No. 19.

U. S. Geological Survey, May 1982. Techniques for Estimating the Magnitude and Frequency of Floods in Dallas-Fort Worth Metropolitan Area, Texas. Water-Resources Investigation Report 82-18.

U. S. Geological Survey, 1997. Regional Equations fro Estimation of Peak-Stream Flow Frequency for Rural Basins in Texas. Water-Resources Investigation Report 96-4307.

Wright-McLaughlin Engineers, 1969. Urban Storm Drainage Criteria Manual. Volumes I and II. Prepared for the Denver Regional Council of Governments, Denver, Colorado

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CHAPTER

3

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CHAPTER

3

HYDRAULIC DESIGN OF STREETS AND CLOSED CONDUITS

Section 3.1
Street and Closed Conduit Design Overview

3.1.1 Storm Water System Design

3.1.1.1 Introduction

Storm water system design is an integral component of both site and overall storm water management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on storm water runoff.

Storm water collection systems must be designed to provide adequate surface drainage while at the same time meeting other storm water management goals such as water quality, streambank protection, habitat protection, and groundwater recharge.

3.1.1.2 System Components

The storm water system components consist of all the *integrated* site design practices and storm water controls utilized on the site. Three considerations largely shape the design of the storm water systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove storm water from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, storm water ponds and wetlands, and small underground pipe systems which collect storm water runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger storm water systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The storm water (major) system consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The storm water system includes not only the trunk line system that receives the water, but also the natural overland relief which functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This chapter is intended to provide design criteria and guidance on several on-site flood control system components, including street and roadway gutters, inlets, and storm drain pipe systems (Section 3.2). Chapter 4 covers the design of culverts (Section 4.2); vegetated and lined open channels (Section 4.4); storage and design (Section 4.5); ; outlet structures (Section 4.6); and energy dissipation devices for outlet protection (Section 4.7). The rest of this section covers important considerations to keep in mind in the planning and design of storm water drainage facilities.

3.1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).

C. Check potential drainage outlets and methods, including:
   1. On-site (structural control, receiving water)
   2. Off-site (highway, storm drain, receiving water, regional control)
   3. Natural drainage system (swales)
   4. Existing drainage system (drain pipe)

D. Analyze areas for probable location of drainage structures and facilities.

E. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall storm water management system and plan.

3.1.2 Key Issues in Storm Water System Design

3.1.2.1 Introduction

The traditional design of storm water systems has been to collect and convey storm water runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Storm water systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural storm water controls to mitigate the major storm water impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in storm water system design.

3.1.2.2 General Design Considerations

- Storm water systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or storm water systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or storm water (minor) systems.
- It is important to ensure that the combined on-site flood control system and major storm water system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor storm water systems and/or major storm water structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of storm water systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major storm water system design capacity.

3.1.2.3 Street and Roadway Gutters

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible, and runoff should be allowed to flow across pervious surfaces or through vegetated channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may be possible not to use gutters at all, or to modify them to channel runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of vegetation, and active maintenance may be necessary in some areas.
- Use typical road sections that use grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. Figure 3.1-1 illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

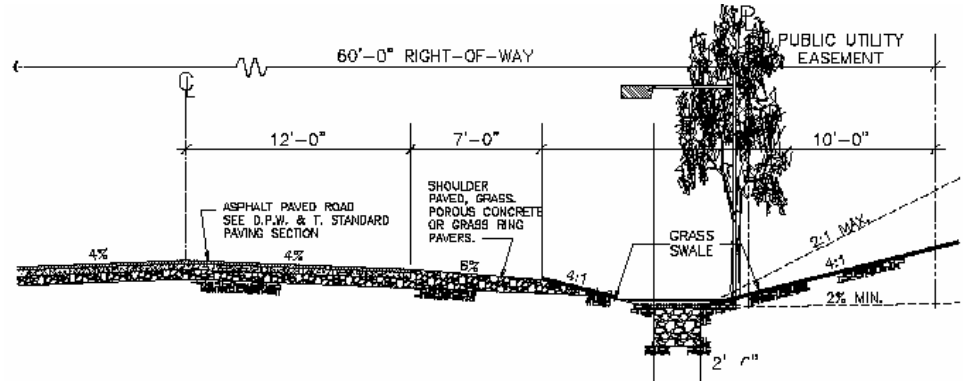


Figure 3.1-1 Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

3.1.2.4 Inlets and Drains

- Inlets should be located to maximize the overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so water flows into vegetated areas prior to entering the nearest inlet.
- Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.
- Inlets should be located to serve as overflows for structural storm water controls. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.
- The choice of inlet type should match its intended use. A sumped inlet may be more effective supporting water quality objectives.
- Use several smaller inlets instead of one large inlet in order to:
  1. Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
  2. Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
  3. Improve aesthetics. Several smaller drains will be less obvious than one large drain.
  4. Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have to travel farther to reach one large drain inlet.

3.1.2.5 Closed Conduit Systems (Storm Drains/Sewers)

- The use of *integrated* site design practices (and corresponding site design credits) should be considered to reduce the overall length of a closed conduit storm water system.
- Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas, natural conservation areas, or filter strips (with spreaders at the end of the pipe).
- Ensure that storms in excess of closed conduit design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The on-site flood control system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

3.1.3 Design Storm Recommendations

Listed below are the design storm recommendations for various storm water drainage system components to be designed and constructed in accordance with the minimum storm water management standards. Some jurisdictions may require the design of storm water conveyance systems, sized for various storm frequencies. Please consult your local review authority to determine the local requirements. It is recommended that the full build-out conditions be used to calculate flows for the design storm frequencies below.

Storm Drainage Systems

Including storm drainage systems and pipes that do not convey runoff under public roadways, sometimes called lateral closed systems.

- 5- to 25-year design storm (for pipe and culvert design)
- 5- to 25-year design storm (for inlet design)
- 100-year design storm (for sumped inlets, unless overflow facilities are provided)

Check Storm

Used to estimate the runoff that is routed through the drainage system and storm water management facilities to determine the effects on the facilities, adjacent property, floodplain encroachment, and downstream areas. The check storm should be contained within public right-of-way and/or easements but should not be conveyed underground in the drainage system.

- 100-year design storm

References

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

Prince George's County, MD, 1999. Low-Impact Development Design Strategies, An Integrated Design Approach.

The Dewberry Companies, 2002, 2nd Edition, Land Development Handbook, McGraw-Hill Companies, Inc., New York, NY.

American Society of Civil Engineers, 1993, Design and Construction of Urban Stormwater Management Systems, Manual and Report No. 77.

Larry W. Mays, Editor, 2001, Stormwater Collection Systems Design Handbook, McGraw-Hill Companies, Inc., New York, NY.

CHAPTER

3

Section 3.2
On-Site Flood Control System Design

3.2.1 Overview

3.2.1.1 Introduction

On-Site Flood Control Systems, also known as minor drainage systems, quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The on-site flood control system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect storm water runoff and transport it to structural control facilities, pervious areas, and/or the larger storm water system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section is intended to provide criteria and guidance for the design of on-site flood control system components including:

- Street and roadway gutters
- Storm water inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in Section 4.4, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb, and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain system design is based on the use of the Rational Method Formula.

3.2.1.2 General Criteria

<u>Design Frequency</u>

See Section 3.1 or the local review authority for design storm requirements for the sizing of storm system components. Fully developed conditions should be used for design.

<u>Flow Spread Limits</u>

Inlets shall be spaced so that the spread in the street for the design flow shall not exceed the local criteria or as a guideline the following, as measured from the face of the curb:

- 8 feet if the street is classified as a Collector or Arterial street (for 4-lane streets or greater, spread may extend across one travel lane)
- 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a Local street

3.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.2-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.2-1 Symbols and Definitions

Symbol	Definition	Units
a	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	-
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L or L_T	Length of curb opening inlet	ft
L	Pipe length	ft
n	Roughness coefficient in the modified Manning's formula for triangular gutter flow	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_i	Intercepted flow	cfs
Q_S	Gutter capacity above the depressed section	cfs
S or S_x	Cross Slope - Traverse slope	ft/ft
S or S_L	Longitudinal slope	ft/ft
S_f	Friction slope	ft/ft
S'_w	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

3.2.3 Street and Roadway Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

3.2.3.1 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56/n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3} \qquad (3.2.1)$$

where:

- Q = gutter flow rate, cfs
- S_x = pavement cross slope, ft/ft
- n = Manning's roughness coefficient
- S = longitudinal slope, ft/ft
- T = width of flow or spread, ft

Equation 3.2.1 was first presented by C.F. Izzard in 1946.

3.2.3.2 Nomograph

Figure 3.2-1 is a nomograph for solving Equation 3.2.1. Manning's n values for various pavement surfaces are presented in Table 3.2-2 below. Note: the nomograph will not work on slopes steeper than 3% - 4% for 2 and 3 lane thoroughfares. Also, the "Q" in the nomograph is only for n = 0.016.

3.2.3.3 Manning's n Table

Table 3.2-2 Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.014
Asphalt pavement: Smooth texture Rough texture	 0.015 0.019
Concrete gutter with asphalt pavement: Smooth Rough	 0.015 0.018
Concrete pavement: Float finish Broom finish	 0.017 0.019
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Note: Based on the statement of Izzard (1946) and confirmation by model studies (Ickert and Crosby, 2003), the n-values given in Table 4-3 of HEC No. 22, 2001, were increased by 18% to derive the n-values in this table.

3.2.3.4 Uniform Cross Slope

The nomograph in Figure 3.2-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not; use the product of Q and n (Qn).

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

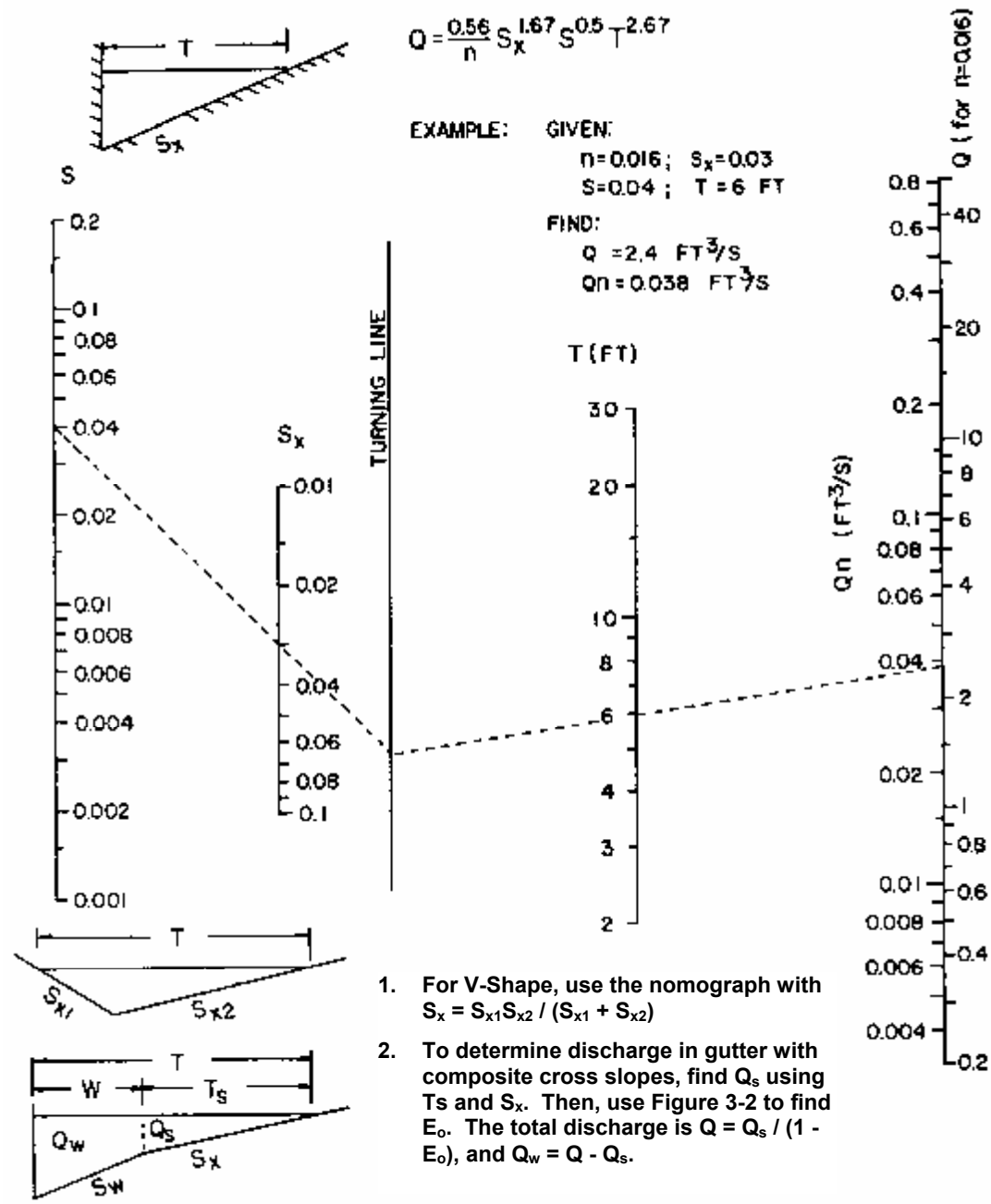
Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.



1. For V-Shape, use the nomograph with $S_x = S_{x1}S_{x2} / (S_{x1} + S_{x2})$
2. To determine discharge in gutter with composite cross slopes, find Q_s using Ts and S_x. Then, use Figure 3-2 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.

Figure 3.2-1 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

***See Section 3.2.3.2 for applicability**

3.2.3.5 Composite Gutter Sections

Figure 3.2-2 in combination with Figure 3.2-1 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Figure 3.2-3 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 3.2-3 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Q_s).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

$$\mathbf{Q_w = Q - Q_s} \qquad \textbf{(3.2.2)}$$

Step 3 Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 3.2-2 to find an appropriate value of W/T.

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6 Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 3.2-1.

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d).

Step 2 Use Figure 3.2-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 3.2-2, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \textbf{(3.2.3)}$$

where:

Q = gutter flow rate, cfs

Q_s = flow capacity of the gutter section above the depressed section, cfs

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

Step 5 Calculate the gutter flow in width (W), using Equation 3.2.2.

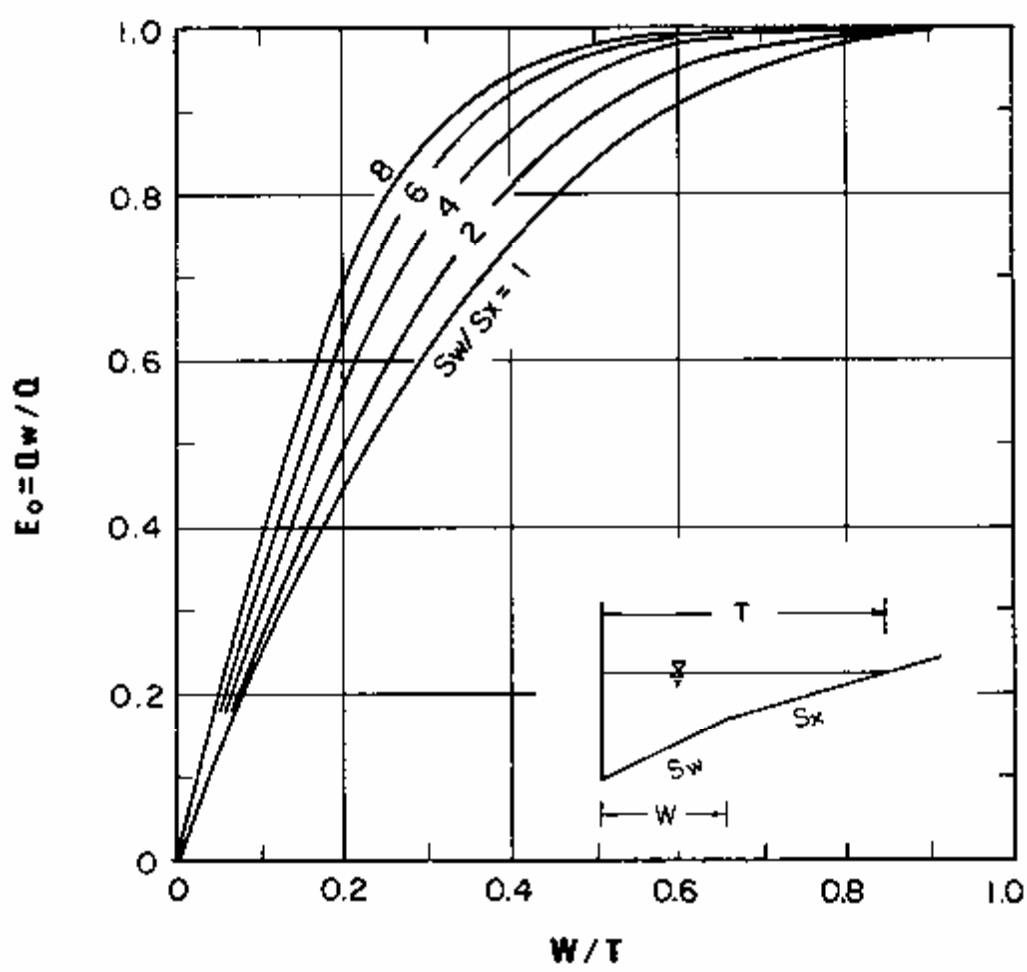


Figure 3.2-2 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

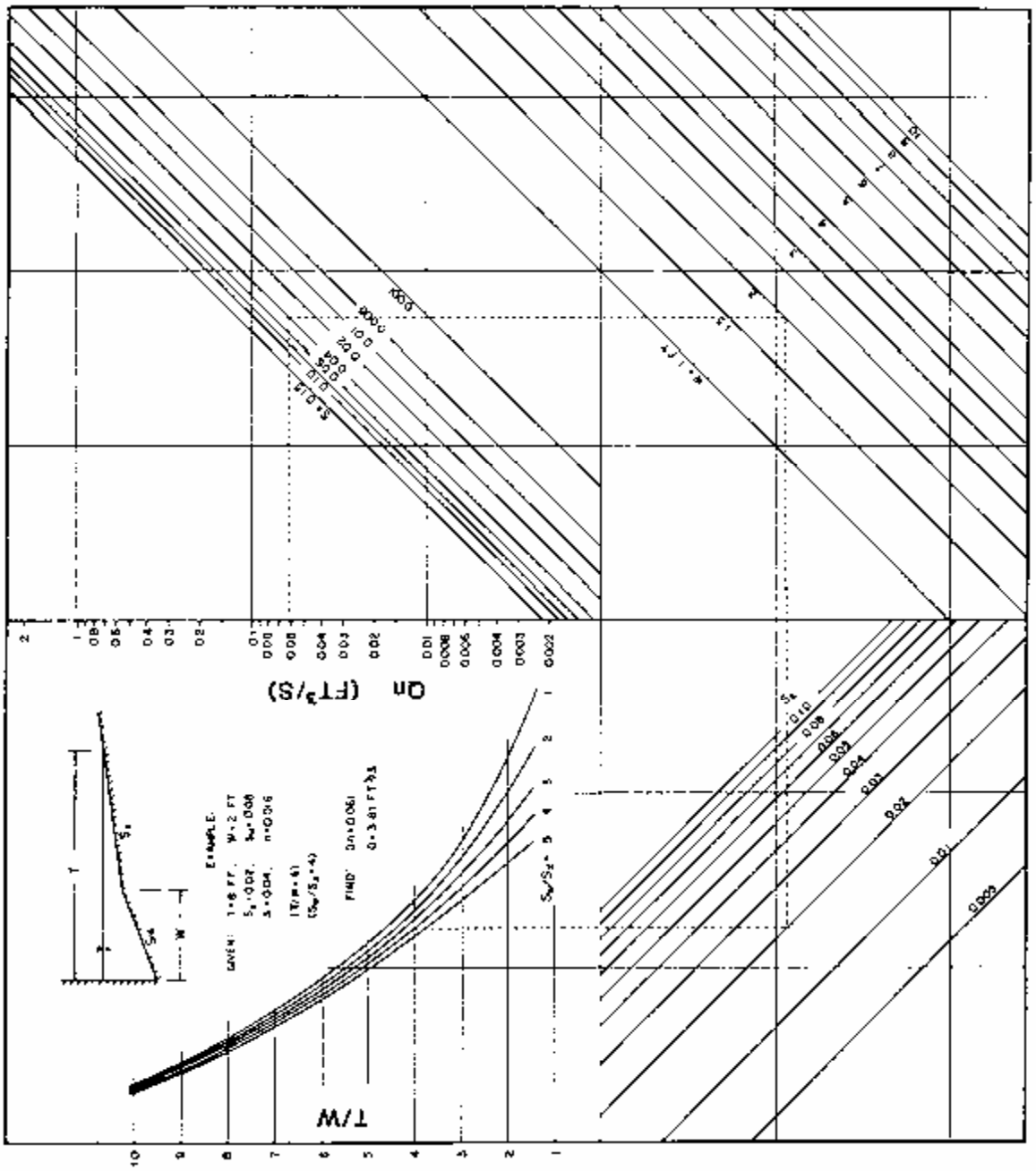


Figure 3.2-3 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

3.2.3.6 Examples

Example 1

Given:

$T = 8$ ft
$S_x = 0.025$ ft/ft
$n = 0.015$
$S = 0.01$ ft/ft

Find:

1. Flow in gutter at design spread
2. Flow in width (W = 2 ft) adjacent to the curb

Solution:

a. From Figure 3.2-1, Qn = 0.03
   Q = Qn/n = 0.03/0.015 = 2.0 cfs

b. T = 8 - 2 = 6 ft
   $(Qn)_2 = 0.014$ (Figure 3.2-1) (flow in 6-foot width outside of width (W))
   Q = 0.014/0.015 = 0.9 cfs
   $Q_w = 2.0 - 0.9 = 1.1$ cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

Given:

$T = 6$ ft
$S_w = 0.0833$ ft/ft
$T_s = 6 - 1.5 = 4.5$ ft
$W = 1.5$ ft
$S_x = 0.03$ ft/ft
$n = 0.014$
$S = 0.04$ ft/ft

Find:

Flow in the composite gutter

Solution:

1. Use Figure 3.2-1 to find the gutter section capacity above the depressed section.
   $Q_s n = 0.038$
   $Q_s = 0.038/0.014 = 2.7$ cfs
2. Calculate W/T = 1.5/6 = 0.25 and
   $S_w/S_x = 0.0833/0.03 = 2.78$
   Use Figure 3.2-2 to find $E_o = 0.64$
3. Calculate the gutter flow using Equation 3.2.3
   Q = 2.7/(1 - 0.64) = 7.5 cfs
4. Calculate the gutter flow in width, W, using Equation 3.2.2
   $Q_W = 7.5 - 2.7 = 4.8$ cfs

3.2.3.7 Parabolic Cross Slope

The following methodology regarding Parabolic cross-slopes was excerpted from the City of Austin Drainage Criteria Manual dated July 2003.

Flows in the gutter of a parabolically crowned pavement are calculated from a variation of Manning's Equation, which assumes steady flow in a prismatic open channel. However, this equation is complicated and difficult to solve for each design case.

To provide a means of determining the flow in the gutter, generalized gutter flow equations for combinations of parabolic crown heights, curb splits and street grades of different street widths have been prepared. All of these equations have a logarithmic form.

Note: The street width used in this section is measured from face of curb to face of curb.

Streets Without Curb Split

Curb split is the vertical difference in elevation between curbs at a given street cross section. The gutter flow equation for parabolic crown streets without any curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 \qquad (3.2.4)$$

where:

Q = Gutter flow rate, cfs

S_0 = Street grade, ft/ft

y_0 = Water depth in the gutter, feet

K_0, K_1, K_2 = Constant coefficients shown in Table 3.2-3 for different street widths

Table 3.2-3 Coefficients for Equation 3.2.4, Streets Without Curb Split

Street Width* (ft)	Coefficients		
	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

***Note:** Based on the Transportation Criteria Manual the street width is measured from face of curb to face of curb (FOC-FOC).

Source: City of Austin, Watershed Engineering Division

Streets With Curb Split

The gutter flow equation for parabolic crown streets with curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3 (CS) \qquad (3.2.5)$$

where:

Q = Gutter flow rate, cfs

S_0 = Street grade, ft/ft

y_0 = Water depth in the gutter, feet

CS = Curb split, feet

K_0, K_1, K_2, K_3 = Constant coefficients shown in Tables 3.2-4 and 3.2-5

The K values in Equation 3.2.5 are found in Tables 3.2-4 and 3.2-5 for different street widths. Table 3.2-4 is used when calculating the higher gutter flows, and Table 3.2-5 is used when calculating the lower gutter flows.

Table 3.2-4 Coefficients for Equation 3.2.5, Streets With Curb Split – Higher Gutter

Street Width* (ft)	Coefficients				Curb Split Range (ft)
	K_0	K_1	K_2	K_3	
30	2.85	0.50	3.03	-0.131	0.0-0.6
36	2.89	0.50	2.99	-0.140	0.0-0.8
40	2.85	0.50	2.89	-0.084	0.0-0.8
44	2.84	0.50	2.83	-0.091	0.0-0.9
48	2.83	0.50	2.78	-0.095	0.0-1.0
60	2.85	0.50	2.74	-0.043	0.0-1.2

Source: City of Austin, Watershed Engineering Division

Table 3.2-5 Coefficients for Equation 3.2.5, Streets With Curb Split – Lower Gutter

Street Width* (ft)	Coefficients				Curb Split Range (ft)
	K_0	K_1	K_2	K_3	
30	2.70	0.50	2.74	-0.215	0.0-0.6
36	2.74	0.50	2.73	-0.214	0.0-0.8
40	2.75	0.50	2.73	-0.198	0.0-0.8
44	2.76	0.50	2.73	-0.186	0.0-0.9
48	2.77	0.50	2.72	-0.175	0.0-1.0
60	2.80	0.50	2.71	-0.159	0.0-1.2

Source: City of Austin, Watershed Engineering Division

All the crown heights for different street widths are calculated by the following equation:

Crown Height (feet) = 0.5 + [(W 30)/120]

where,

W = street width, feet

Parabolic Crown Location

The gutter flow equation presented for parabolic crowns with split curb heights is based on a procedure for locating the street crown. The procedure allows the street crown to shift from the street center line toward the high ¼ point of the street in direct proportion to the amount of curb split. The maximum curb split occurs with the crown at the ¼ point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 feet per foot of street width, except for 30-foot side streets where the maximum curb split is considered to be 5 feet..

Example: Determination of Crown Location

Given: 0.4 feet Design split on 30-foot wide street.

Maximum Curb Split = 0.02 x street width
= 0.2 x 30 feet = 0.6 feet
Maximum Movement = ¼ street width for 30-foot street
= ¼ x 30 feet = 7.5 feet

Split Movement = (Design Split x W/Maximum Split x 4)
= (0.4 x 30/0.6 x 4) = 5 feet

Curb splits that are determined by field survey, whether built intentionally or not, should be considered when determining the capacity of the curb flow.

Special consideration should be given when working with cross sections which have the pavement crown above the top of curb. When the crown exceeds the height of the curb the maximum depth of water is equal to the height of the curb, not the crown height. It should be noted that a parabolic section where the crown equals the top of curb will carry more water than a section which has the crown one (1) inch above the top of curb. For other parabolic roadway sections not included in Tables 3.2-3, 3.2-4, and 3.2-5, see HEC-22 Urban Design Manual by the Federal Highway Administration, dated August 2001.

3.2.4 Storm Water Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load-bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions. Sump areas should have an overflow route or channel.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so they will limit spread on low gradient

approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in subsection 3.2.5, curb inlet design in Section 3.2.6, and combination inlets in Section 3.2.7.

3.2.5 Grate Inlet Design

3.2.5.1 Grate Inlets on Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 3.2-3 presents the results of debris handling efficiencies of several grates. Debris handling efficiencies were based on the total number of simulated leaves arriving at the grate and the number passed.

The ratio of frontal flow to total gutter flow, E_O, for straight cross slope is expressed by the following equation:

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \qquad (3.2.6)$$

where:

Q = total gutter flow, cfs

Q_W = flow in width W, cfs

W = width of depressed gutter or grate, ft

T = total spread of water in the gutter, ft

Table 3.2-6 Grate Debris Handling Efficiencies			
Rank	Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	CV - 3-1/4 - 4-1/4	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Reticuline	12	16
8	P - 1-1/8	9	20

Source: "Drainage of Highway Pavements" (HEC-12), Federal Highway Administration, 1984.

Figure 3.2-2 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \quad (3.2.7)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\,(V - V_0) \quad (3.2.8)$$

where:

V = velocity of flow in the gutter, ft/s (using Q from Figure 3.2-1)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 3.2-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 3.2-4 provides a solution of equation 3.2.8, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3.2-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \quad (3.2.9)$$

where:

L = length of the grate, ft

Figure 3.2-5 provides a solution to equation 3.2.9.

The efficiency, E, of a grate is expressed as:

$$E = R_fE_o + R_s(1 - E_o) \quad (3.2.10)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)] \quad (3.2.11)$$

The following example illustrates the use of this procedure.

Given:

W = 2 ft

T = 8 ft

S_x = 0.025 ft/ft

$S = 0.01$ ft/ft
$E_o = 0.69$
$Q = 3.0$ cfs
$V = 3.1$ ft/s
Gutter depression = 2 in

Find:

Interception capacity of:

1. a curved vane grate, and
2. a reticuline grate 2-ft long and 2-ft wide

Solution:

From Figure 3.2-4 for Curved Vane Grate, $R_f = 1.0$
From Figure 3.2-4 for Reticuline Grate, $R_f = 1.0$
From Figure 3.2-5 $R_s = 0.1$ for both grates
From Equation 3.2.11:

$Q_i = 3.0[1.0 \text{ X } 0.69 + 0.1(1 - 0.69)] = 2.2$ cfs

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

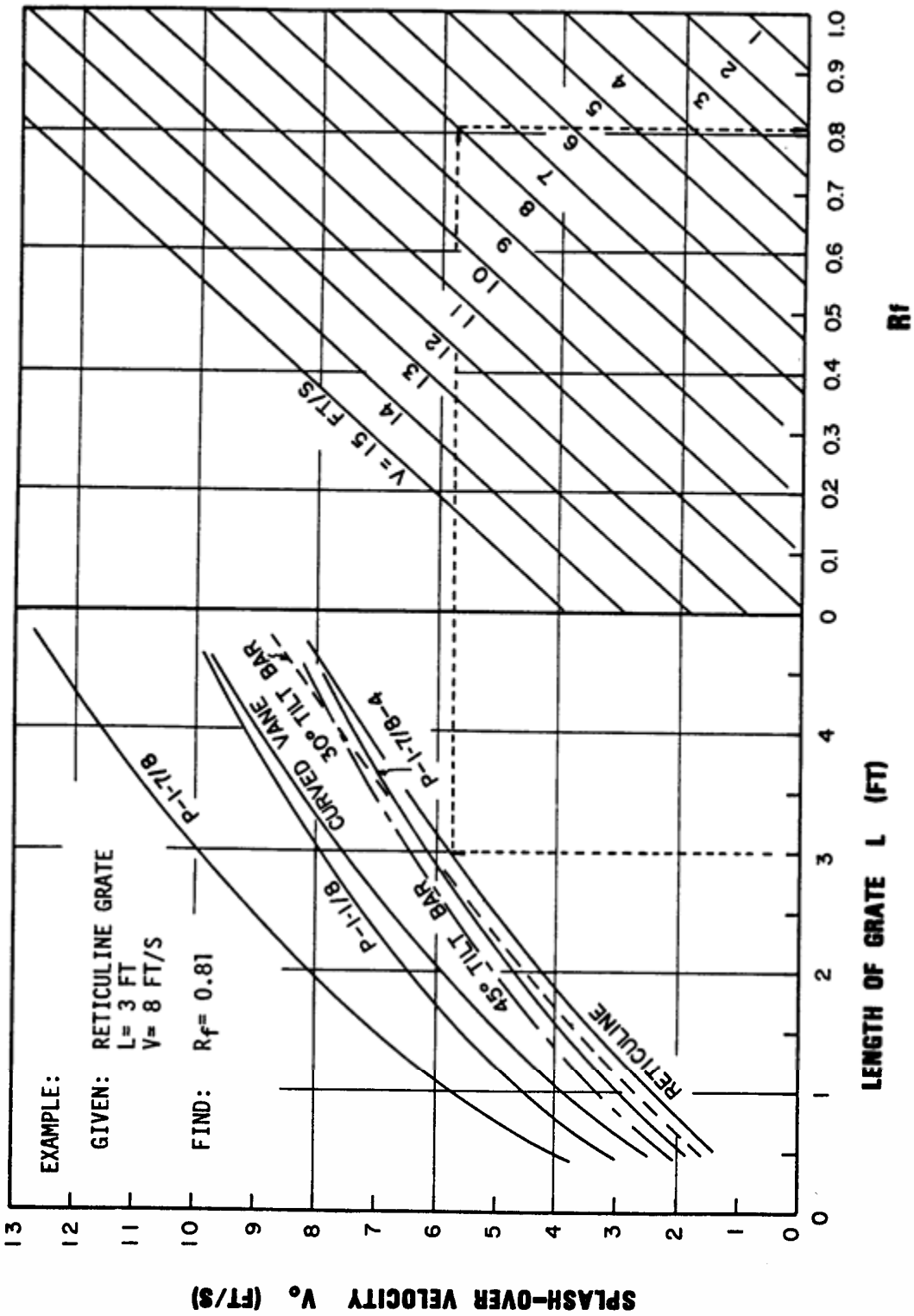


Figure 3.2-4 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

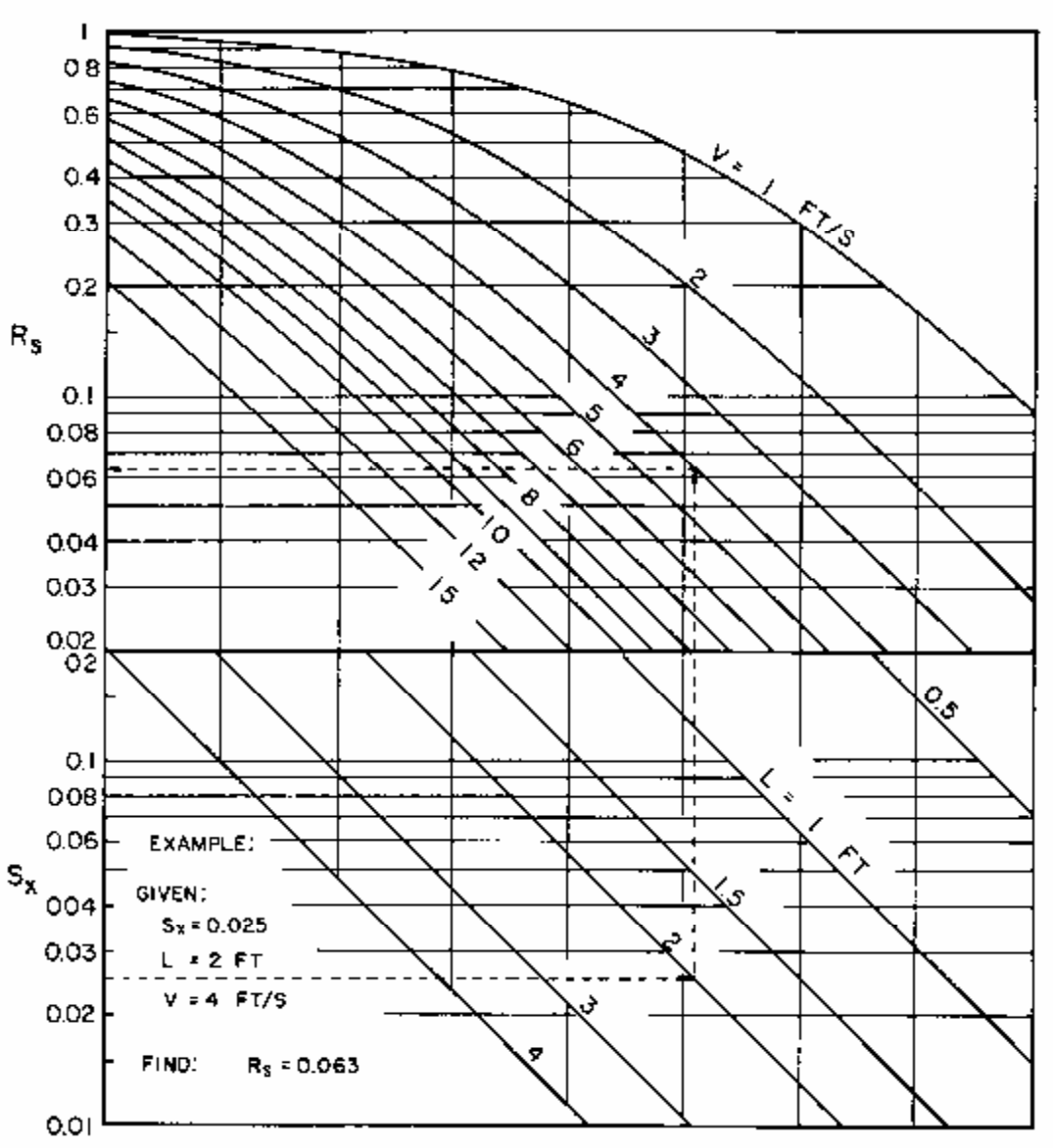


Figure 3.2-5 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

3.2.5.2 Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$Q_i = CPd^{1.5} \qquad (3.2.12)$$

where:

P = perimeter of grate excluding bar widths and the side against the curb, ft

C = 3.0

d = depth of water above grate, ft

and as an orifice is:

$$Q_i = CA(2gd)^{0.5} \qquad (3.2.13)$$

where:

C = 0.67 orifice coefficient

A = clear opening area of the grate, ft^2

g = 32.2 ft/s^2

Figure 3.2-6 is a plot of equations 3.2.12 and 3.2.13 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given:

A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

Q_b = 3.6 cfs

Q = 8 cfs, 25-year storm

T = 10 ft, design

S_x = 0.05 ft/ft

d = TS_x = 0.5 ft

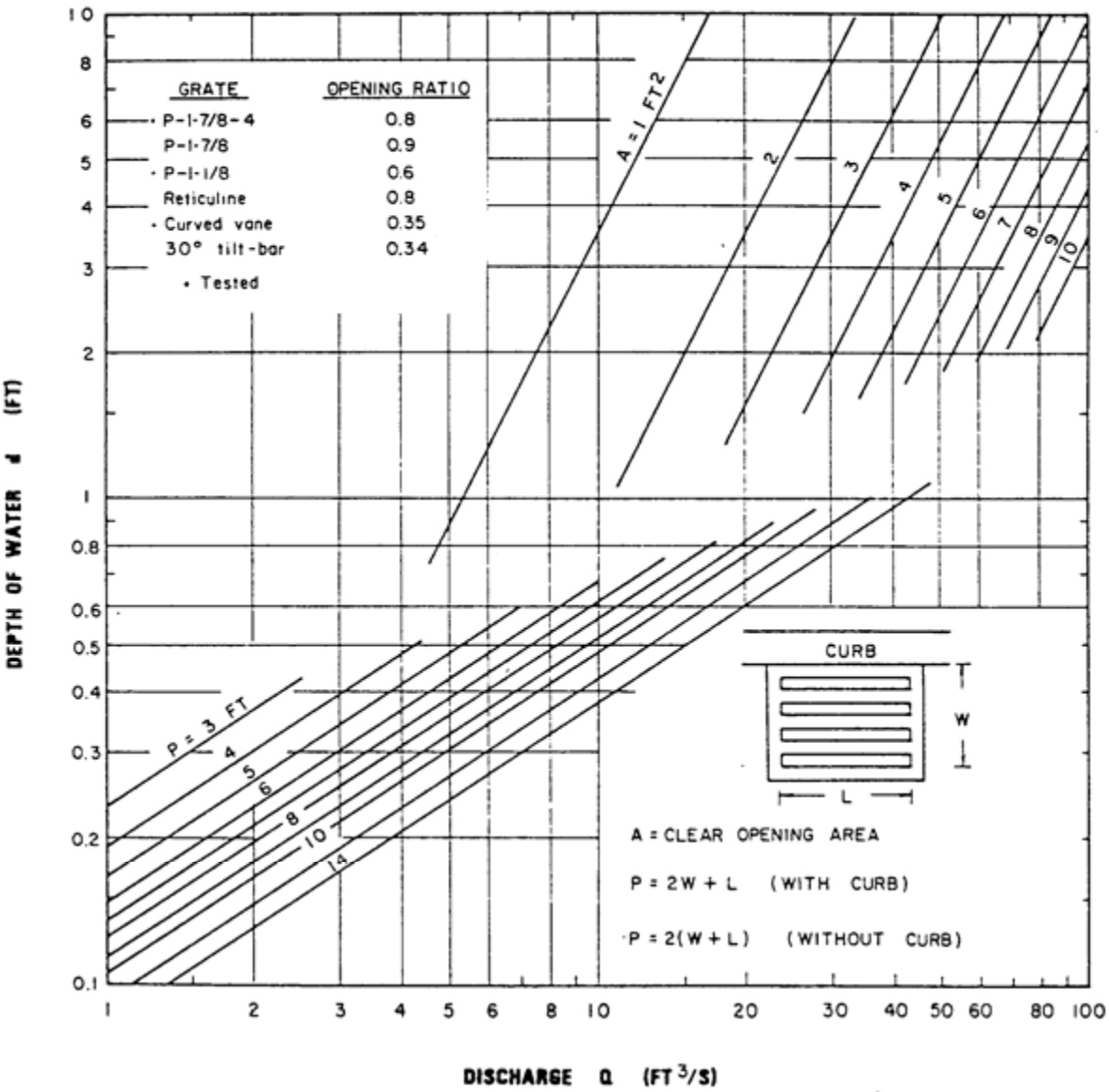
Find:

Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

Solution:

From Figure 3.2-6, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%.



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 3.2-6 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50% clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from Figure 3.2-6)

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 3.2-1)

Thus a double 2 x 3-ft grate inlet with 50% clogging is adequate to intercept the design flow at a spread that does not exceed design spread, and to ensure the spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet or curb-opening inlet in sag where ponding can occur, as well as flanking inlets on the low gradient approaches.

3.2.6 Curb Inlet Design

3.2.6.1 Curb Inlets on Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 3.2-7. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 3.2-8.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in the following equation:

$$\mathbf{S_e = S_x + S'_w E_o} \qquad \textbf{(3.2.14)}$$

where:

E_o = ratio of flow in the depressed section to total gutter flow

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x

S'_w = (a/12W)

where:

a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 3.2-7 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

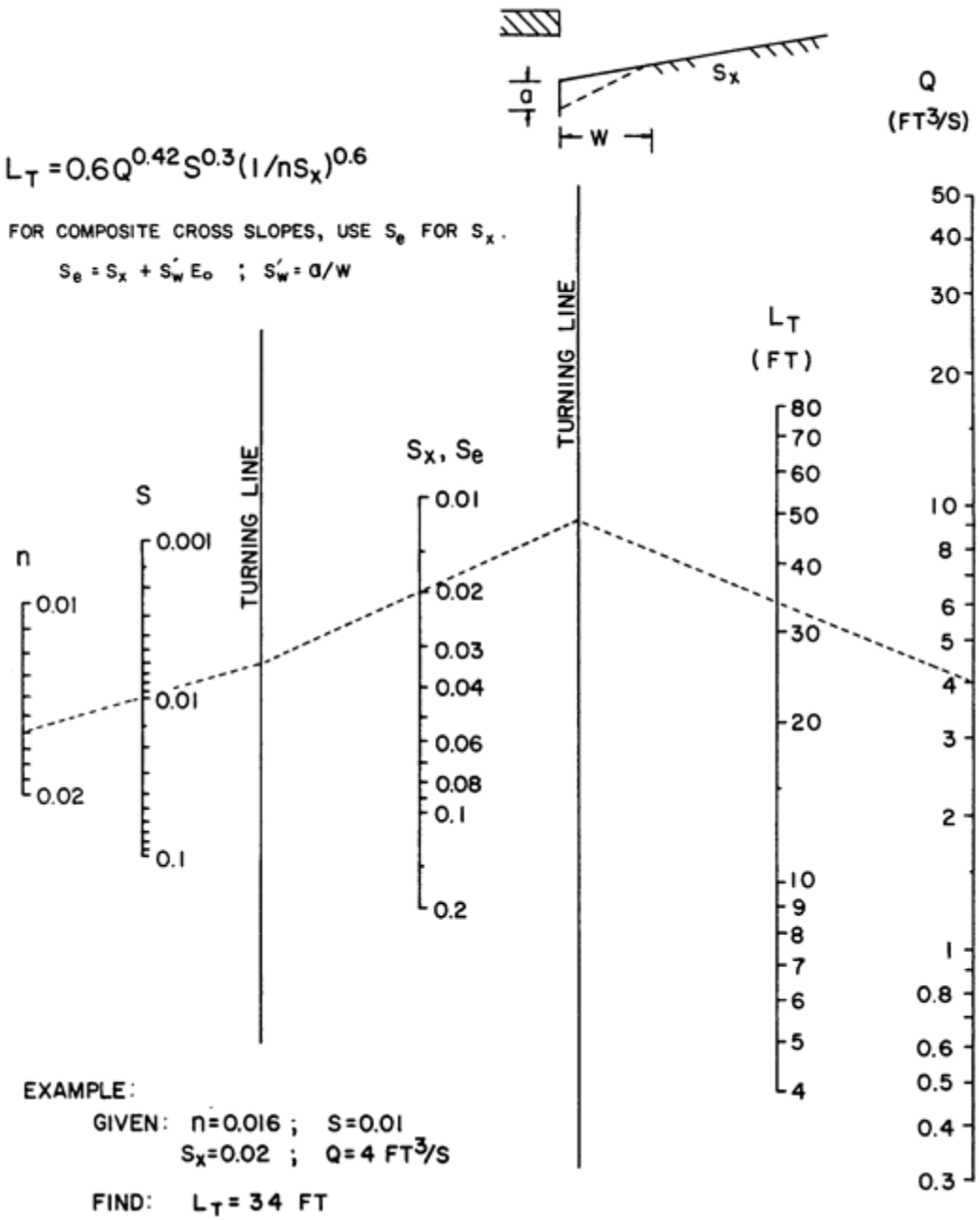


Figure 3.2-7 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

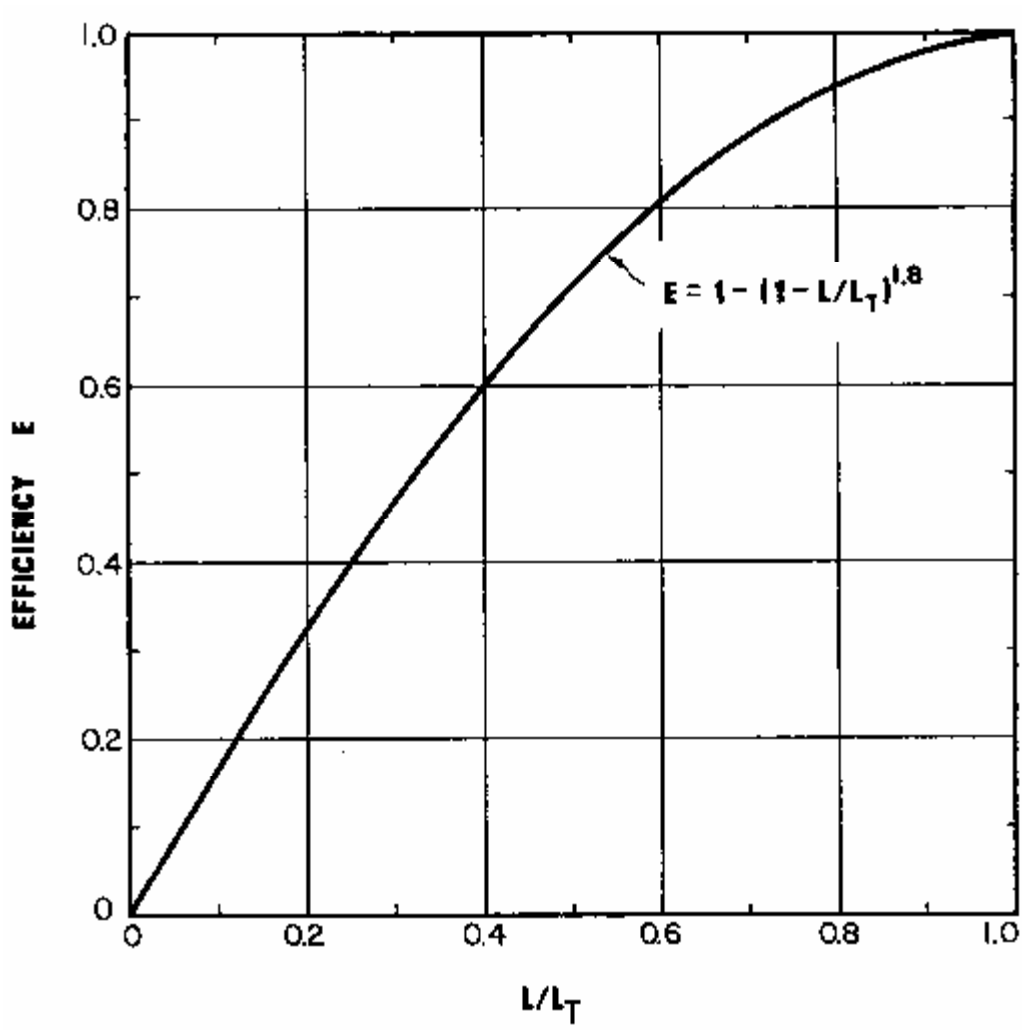


Figure 3.2-8 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

Design Steps

Steps for using Figures 3.2-7 and 3.2-8 in the design of curb inlets on grade are given below.

Step 1 Determine the following input parameters:

Cross slope = S_x (ft/ft)

Longitudinal slope = S (ft/ft)

Gutter flow rate = Q (cfs)

Manning's n = n

Spread of water on pavement = T (ft) from Figure 3.2-1

Step 2 Enter Figure 3.2-7 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

Step 3 Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.

Step 4 Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length required from the vertical line labeled L_T.

Step 5 If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 3.2-8 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

Example

Given:

S_x = 0.03 ft/ft

n = 0.016

S = 0.035 ft/ft

Q = 5 cfs

S'_w = 0.083 (a = 2 in, W = 2 ft)

Find:

1. Q_i for a 10-ft curb-opening inlet
2. Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 3.2-1)

Solution:

1. From Figure 3.2-7, L_T = 41 ft, L/L_T = 10/41 = 0.24
   From Figure 3.2-8, E = 0.39, Q_i = EQ = 0.39 x 5 = 2 cfs
2. Qn = 5.0 x 0.016 = 0.08 cfs
   S_w/S_x = (0.03 + 0.083)/0.03 = 3.77
   T/W = 3.5 (from Figure 3.2-3)
   T = 3.5 x 2 = 7 ft
   W/T = 2/7 = 0.29 ft
   E_o = 0.72 (from Figure 3.2-2)
   Therefore, $S_e = S_x + S'_wE_o$ = 0.03 + 0.083(0.72) = 0.09
   From Figure 3.2-7, L_T = 23 ft, L/L_T = 10/23 = 0.43
   From Figure 3.2-8, E = 0.64, Q_i = 0.64 x 5 = 3.2 cfs

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

3.2.6.2 Curb Inlets in Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The capacity of curb-opening inlets in a sump location can be determined from Figure 3.2-9, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure 3.2-9). The weir portion of Figure 3.2-9 is valid for a depressed curb-opening inlet when $d \leq (h + a/12)$.

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 3.2-10. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 3.2-11.

Design Steps

Steps for using Figures 3.2-9, 3.2-10, and 3.2-11 in the design of curb-opening inlets in sump locations are given below.

Step 1 Determine the following input parameters:
Cross slope = S_x (ft/ft)
Spread of water on pavement = T (ft) from Figure 3.2-1
Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]
Dimensions of depression if any [a (in) and W (ft)]

Step 2 To determine discharge given the other input parameters, select the appropriate figure (3.2-9, 3.2-10, or 3.2-11 depending on whether the inlet is in a depression and if the orifice opening is vertical).

Step 3 To determine the discharge (Q), given the water depth (d), locate the water depth value on the y-axis and draw a horizontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

Step 4 To determine the water depth given the discharge, use the procedure described in Step 3 except enter the figure at the value for the discharge on the x-axis.

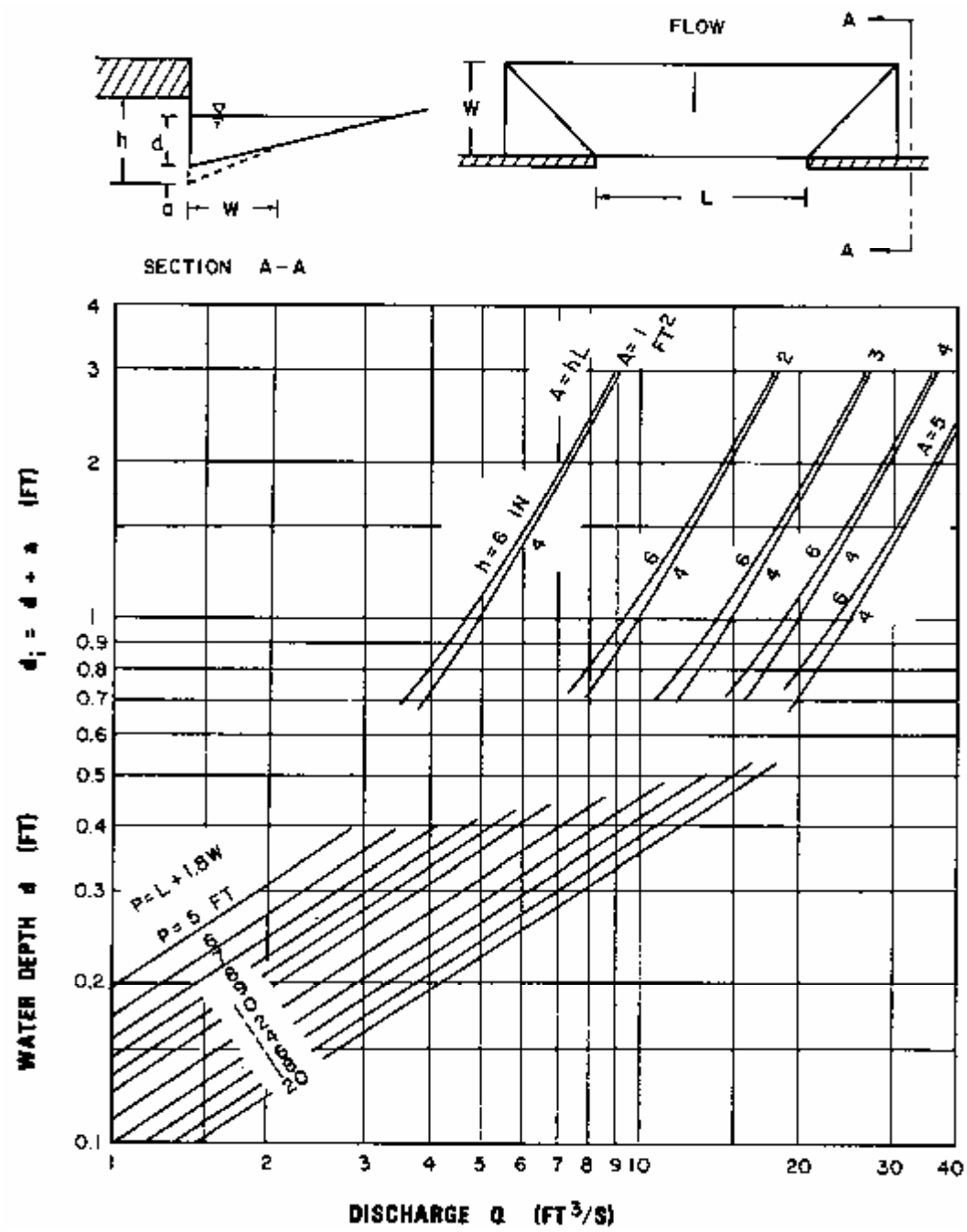


Figure 3.2-9 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

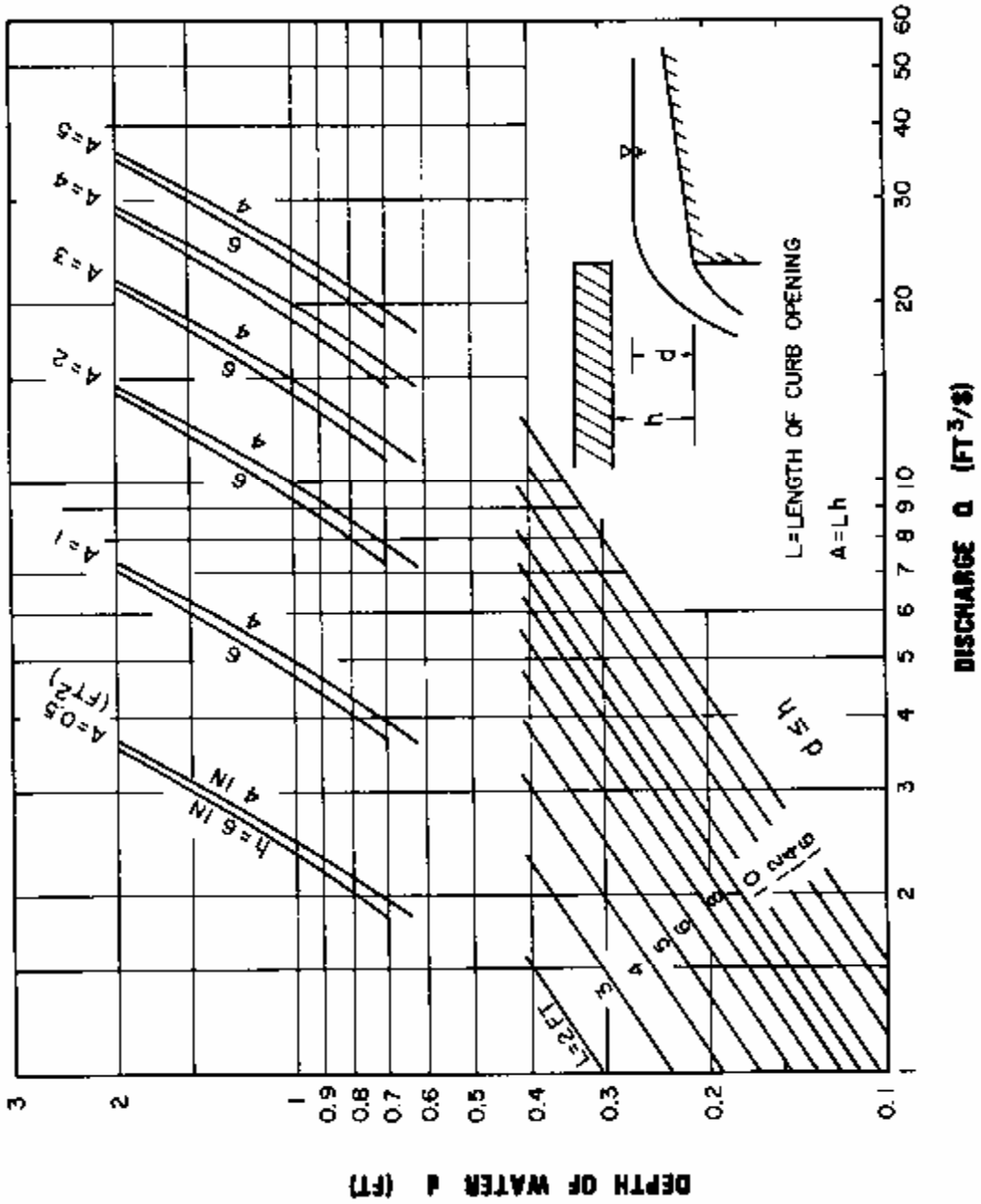


Figure 3.2-10 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

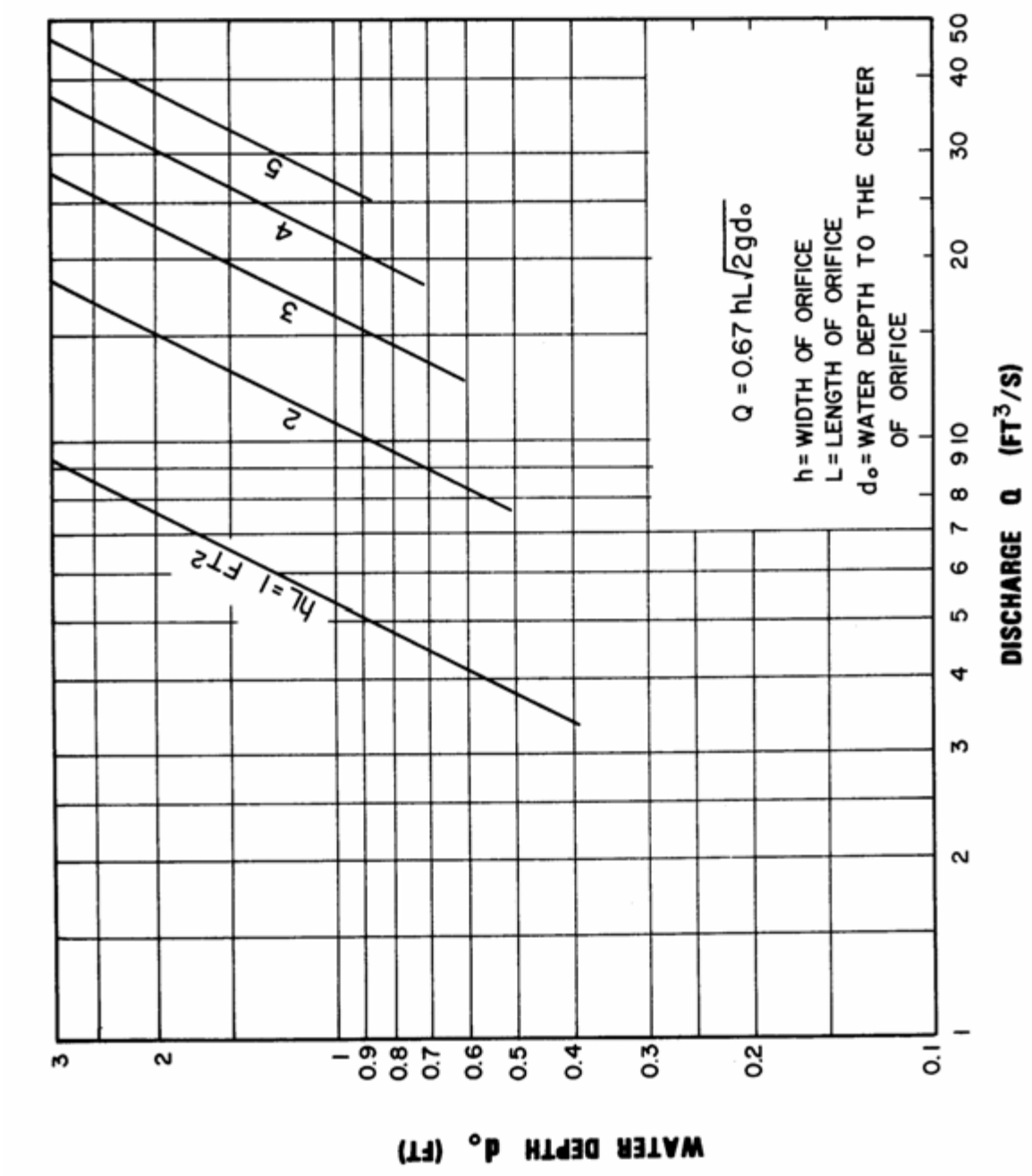


Figure 3.2-11 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

Example:"
Given:

Curb-opening inlet in a sump location

L = 5 ft

h = 5 in

1. Undepressed curb opening
   S_x = 0.05 ft/ft
   T = 8 ft
2. Depressed curb opening
   S_x = 0.05 ft/ft
   a = 2 in
   W = 2 ft
   T = 8 ft

Find:

Discharge Q_i

Solution:

1. d = TS_x = 8 x 0.05 = 0.4 ft
   d < h
   From Figure 3.2-10, Q_i = 3.8 cfs
2. d = 0.4 ft
   h + a/12 = (5 + 2/12)/12 = 0.43 ft

   since d < 0.43 the weir portion of Figure 3.2-9 is applicable (lower portion of the figure).

   P = L + 1.8W = 5 + 3.6 = 8.6 ft
   From Figure 3.2-9, Q_i = 5 cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

3.2.7 Combination Inlets

3.2.7.1 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 3.2-4, 3.2-5, and 3.2-6.

3.2.7.2 Combination Inlets in Sump

All debris carried by storm water runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 3.2-9, 3.2-10, and 3.2-11 for curb-opening inlets should be used for design.

3.2.8 Closed Conduit Systems (Storm Drains/Sewers)

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at other structural storm water controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

Closed conduit system are composed of different lengths and sizes of conduits (system segments) connected by appointment structures (system nodes). Segments are most often circular pipe, but can be a box or other enclosed conduit. Materials used are usually corrugated metal, plastic, and concrete but may be of other materials.

Appurtenant structures serve many functions. Inlets, access holes, and junction chambers are presented in sections 3.2.8.1 through 3.2.8.3.

3.2.8.1 Inlets

The primary function is to allow surface water to enter the closed conduit system. Inlet structures may also serve as access points for cleaning and inspection. Typical inlets structures are a standard drop inlet, catch basin, curb inlet, combination inlet, and Y inlet. (See Figures 3.2-12 and 3.2-13).

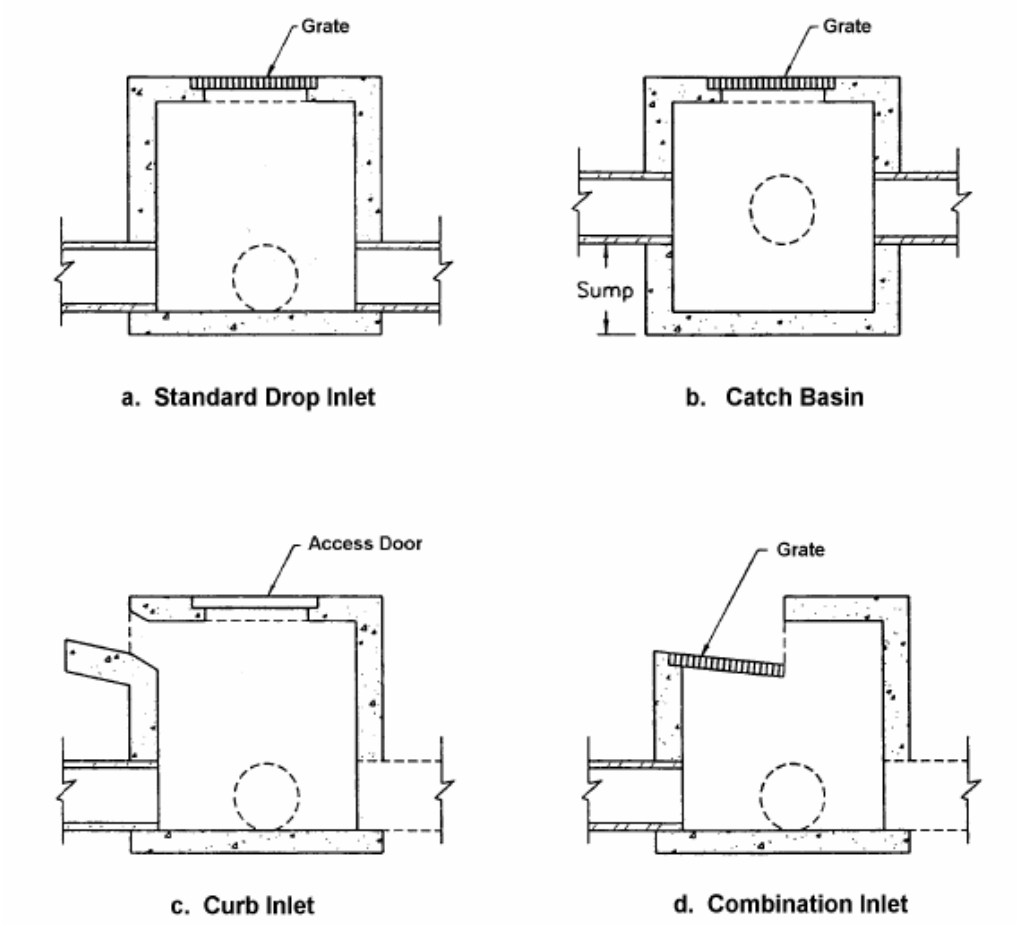


Figure 3.2-12 Inlet Structures
(HEC 22, 2001)

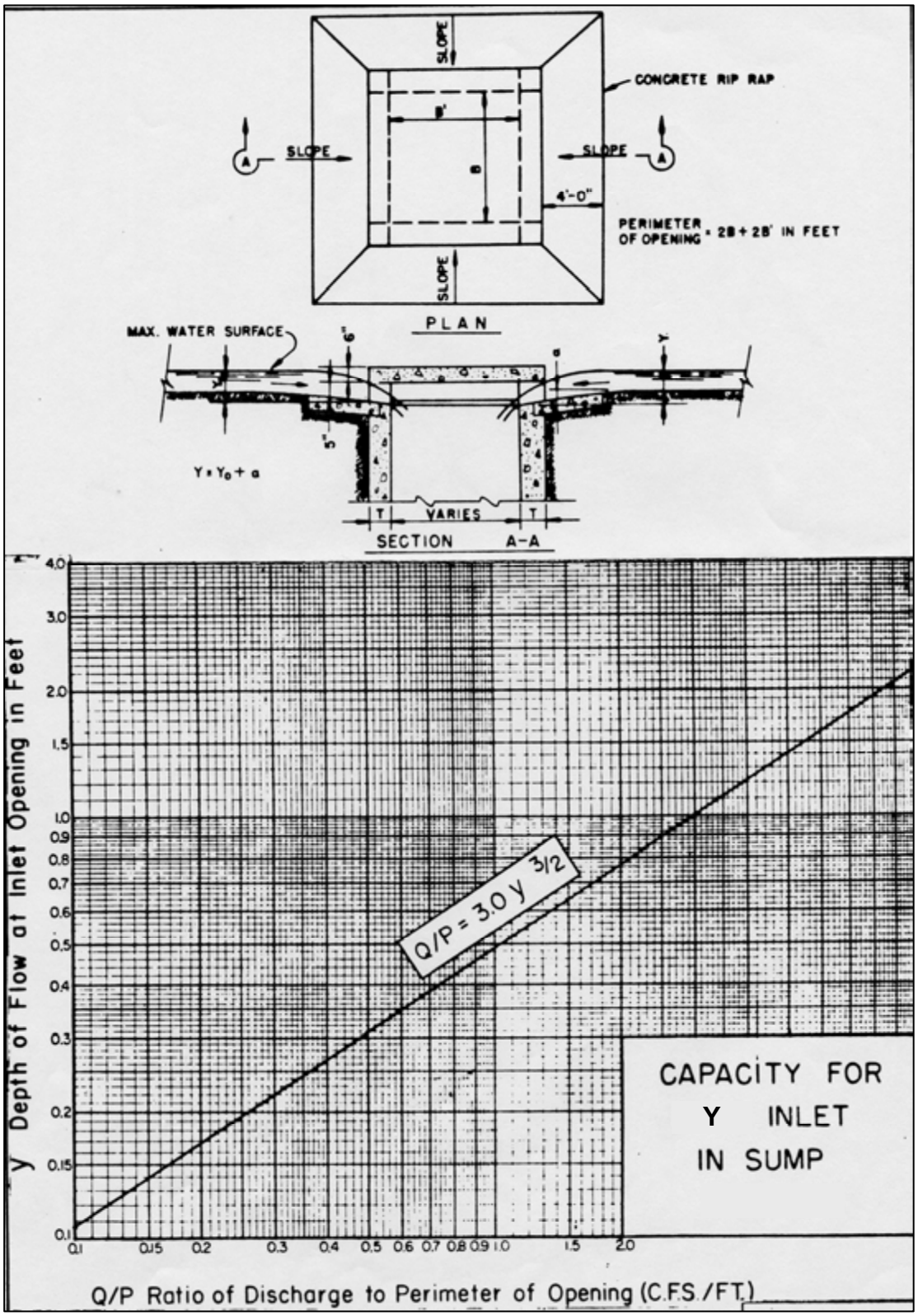


Figure 3.2-13 Capacity for Y Inlet in Sump
(Fort Worth, 1967)

Inlet structures are located at the upstream end and at intermediate points within the closed conduit system. Inlet placement is generally a trial and error procedure that attempts to produce the most economical and hydraulically effective system (HEC 22, 2001).

3.2.8.2 Access Holes (Manholes)

The primary function of an access hole is to provide access to the closed conduit system. An access hole can also serve as a flow junction and can provide ventilation and pressure relief. Typical access holes are shown in Figures 3.2-14 and 3.2-15 (HEC 22, 2001). The materials commonly used for access hole construction are precast concrete and cast-in-place concrete.

Spacing criteria are typically established by local agencies. At a minimum, access holes should be located at the following points:

- Where two or more storm drains converge
- Where pipe sizes change
- Where a change in alignment occurs
- Where a change in grade occurs

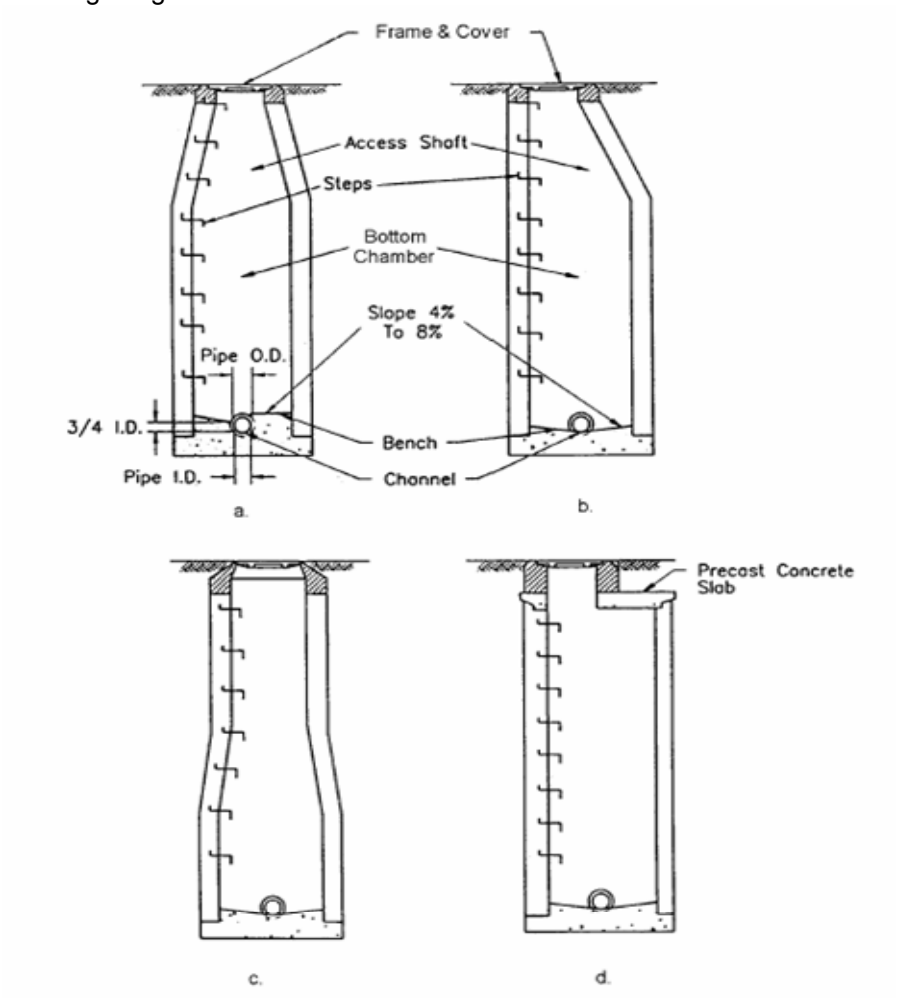


Figure 3.2-14 Typical Access Hole Configurations.
(HEC22, 2001)

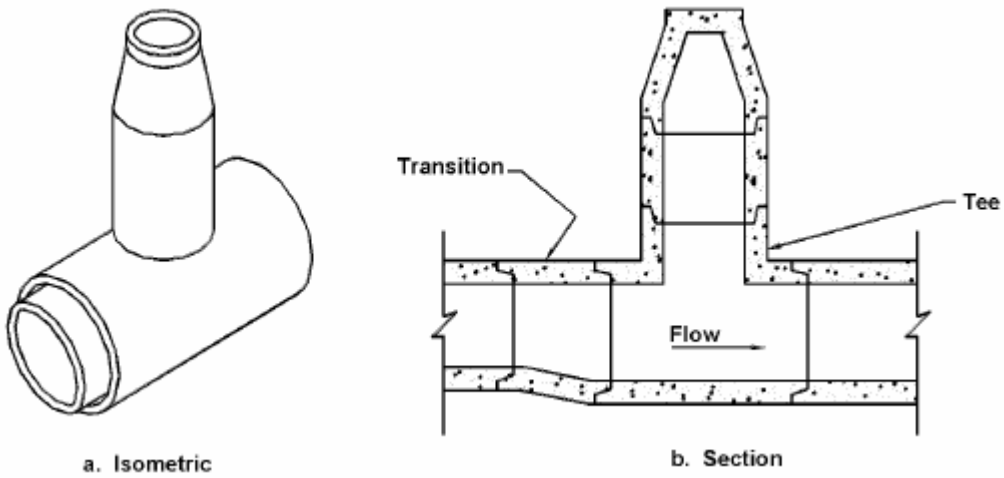


Figure 3.2-15 "Tee" Access Hole for Large Storm Drains
(HEC 22, 2001)

Access holes may be needed at intermediate points along straight runs of closed conduits. Table 3.2-7 gives suggested maximum spacing criteria.

Table 3.2-7 Access Hole Spacing Criteria
(HEC 22, 2001)

Pipe Size (inches)	Suggested Maximum Spacing (feet)
12-24	300
27-36	400
42-54	500
60 and up	1000

3.2.8.3 Junction Chambers

A junction chamber, or junction box, is a special design underground chamber used to join two or more large storm drain conduits. This type of structure is usually required where storm drains are larger than the size that can be accommodated by standard access holes. For smaller diameter storm drains, access holes are typically used instead of junction chambers. Junction chambers by definition do not need to extend to the ground surface and can be completely buried. However, it is recommended that riser structures be used to provide surface access and/or to intercept surface runoff.

Materials commonly used for junction chamber construction include pre-cast concrete and cast-in-place concrete. On storm drains constructed of corrugated steel, the junction chambers are sometimes made of the same material.

To minimize flow turbulence in junction boxes, flow channels and benches are typically built into the bottom of the chambers. Where junction chambers are used as access points for the storm drain system, their location should adhere to the spacing criteria outlined in Table 3.2-7.

3.2.8.4 Design Criteria

Specific design criteria will likely vary from community to community. In the design of closed conduit systems, the following are offered for consideration in setting local criteria:

- For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.
- The maximum hydraulic gradient should not produce a velocity that exceeds 15 ft/s.
- The minimum desirable physical slope should be 0.5% or the slope that will produce a velocity of 2.5 feet per second when the storm sewer is flowing full, whichever is greater.
- If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

3.2.8.5 General Design Procedure

The design of storm drain systems generally follows these steps:

Step 1 Determine inlet location and spacing as outlined earlier in this section.

Step 2 Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

Step 3 Determine drainage areas and compute runoff using the Rational Method

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from the upstream end of a line downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (Figure 3.2-25) can be used to summarize hydrologic, hydraulic and design computations.

Step 5 Examine assumptions to determine if any adjustments are needed to the final design.

The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less that this total. The Rational Method is the most common means of determining design discharges for storm drain design. The time of concentration is very influential in the determination of the design discharge using the Rational Method. The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of interest. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration for inlet spacing is the time required for water to flow from the hydraulically most distant point of the unique drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

The time of concentration for pipe sizing is defined as the time required for water to travel for the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest.

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the higher runoff coefficient (C value) and higher intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighed C value and the intensity associated with the longest time of concentration.
- Secondly, calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

- First, calculate the runoff form the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.
- Secondly, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The portion of the larger primary area to be considered is determined by the following: $A_c=A\ (t_{c1}/t_{c2})$.

A_c is the most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area. A is the area of the larger primary area, t_{c1} is the time of concentration of the smaller, less pervious, tributary area, and t_{c2} is the time of concentration associated with the larger primary area as is used in the first calculation. The C value to be used in this computation should be the weighted C value of the smaller less pervious tributary area and the area A_c. The area to be used in the Rational Method would be the area of the less pervious area plus A_c. The second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, the results of these calculations should be compared and the largest value of discharge should be used for design.

3.2.8.6 Capacity Calculations

The design procedures presented here assume flow within each storm drain segment is steady and uniform. This means the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

Although at times flow in a closed conduit may be under pressure or at other times the conduit may flow partially full, the usual design assumption is that the conduit is flowing fill but not under pressure. Under this assumption the rate of head loss is the same as the slope of the pipe (S_f=S, ft/ft). Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow.

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$V = (1.486/n)\ R^{2/3}\ S^{1/2} \qquad (3.2.15)$$

where:

- V = mean velocity of flow, ft/s
- R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
- S = the slope of hydraulic grade line, ft/ft
- n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$Q = (1.486/n)\ A\ R^{2/3}\ S^{1/2} \qquad (3.2.16)$$

where:

- Q = rate of flow, cfs
- A = cross sectional area of flow, ft^2

For pipes flowing full, the area is $(\pi/4)D^2$ and the hydraulic radius is D/4, so, the above equations become:

$$V = [0.590\ D^{2/3}S^{1/2}]/n \qquad (3.2.17)$$

$$Q = [0.463\ D^{8/3}S^{1/2}]/n \qquad (3.2.18)$$

where:

- D = diameter of pipe, ft
- S = slope of the pipe = S_f hydraulic grade line, ft/ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [0.453\ n^2V^2L]/[R^{4/3}] \qquad (3.2.19)$$

$$H_f = [(2.87\ n^2V^2L]/[D^{4/3}] \qquad (3.2.20)$$

$$H_f = [(185n^2(V^2/2g)L]/[D^{4/3}] \qquad (3.2.21)$$

where:

H_f = total head loss due to friction, ft (S_f x L)
n = Manning's roughness coefficient
D = diameter of pipe, ft
L = length of pipe, ft
V = mean velocity, ft/s
R = hydraulic radius, ft
g = acceleration of gravity = 32.2 ft/sec^2

A nomograph solution of Manning's Equation for full flow in circular conduits is presented in Figure 3.2-16. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 3.2-9. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for conduits may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

Figure 3.2-17 illustrates storm drain capacity sensitivity to the parameters in the Manning's equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased by a factor of 6.0; if the slope is doubled, the capacity is increased by a factor of 1.4; however, if the roughness is doubled, the pipe capacity will be reduced by 50 percent.

The hydraulic elements graph in Figures 3.2-18a and 3.2-18b is provided to assist in the solution of the Manning's equation for part full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

1. Peak flow occurs at 93 percent of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
2. The velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
4. As the depth of flow drops below half-full, the flow velocity drops off rapidly. The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape. Table 3.2-8 provides a tabular listing of the increase in capacity which can be achieved using alternate conduit shapes that have the same height as the original circular shape, but have a different cross sectional area. Although these alternate shapes are generally more expensive than circular shapes, their use can be justified in some instances based on their increased capacity.

Table 3.2-8 Increase in Capacity of Alternate Conduit Shapes Based on a Circular Pipe with the Same Height
(HEC-22, 2001)

	Area (Percent Increase)	Conveyance (Percent Increase)
Circular	--	--
Oval	63	87
Arch	57	78
Box (B = D)	27	27

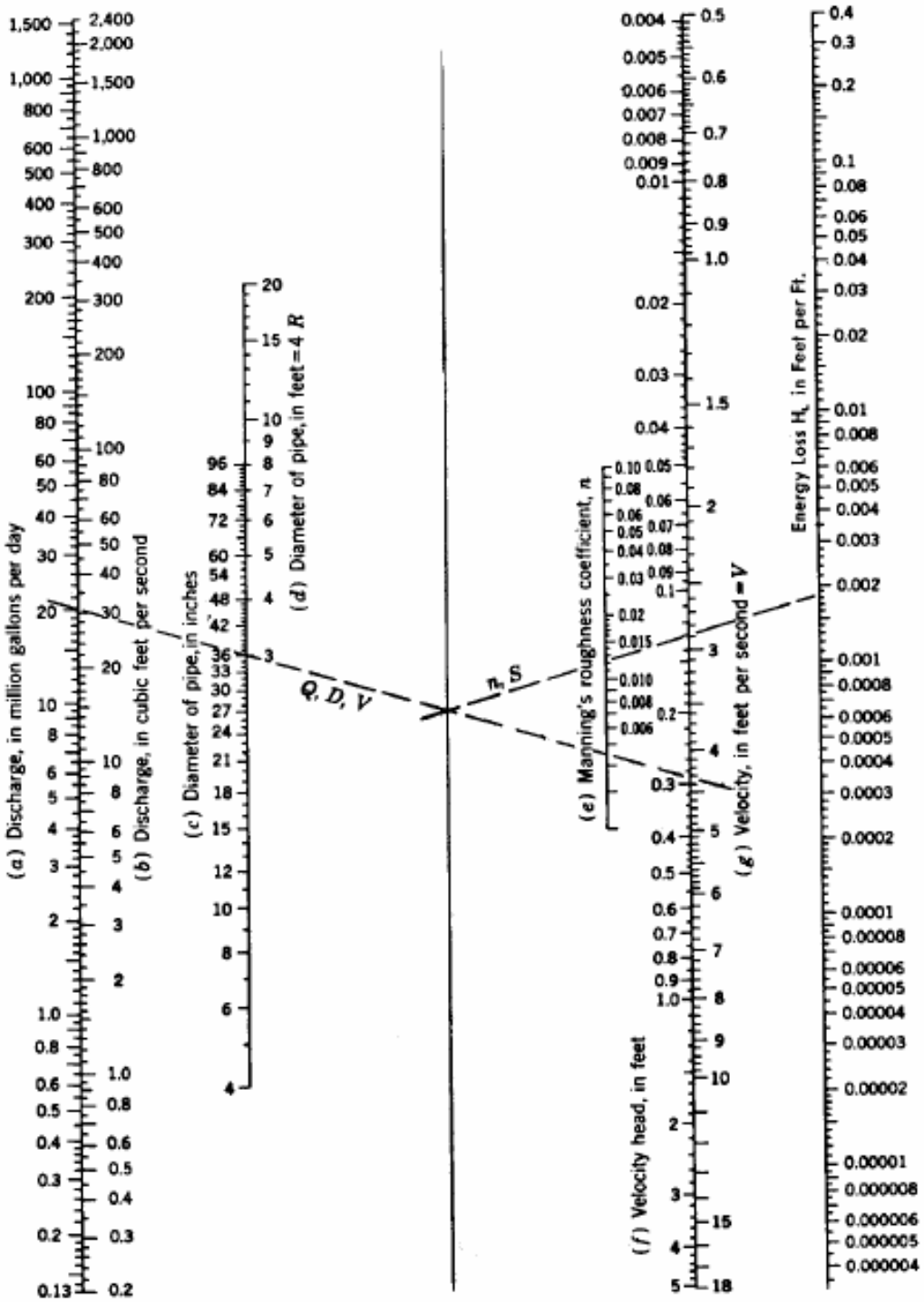


Figure 3.2-16 Solution of Manning's Equation for Flow in Storm Drains-English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Institute)

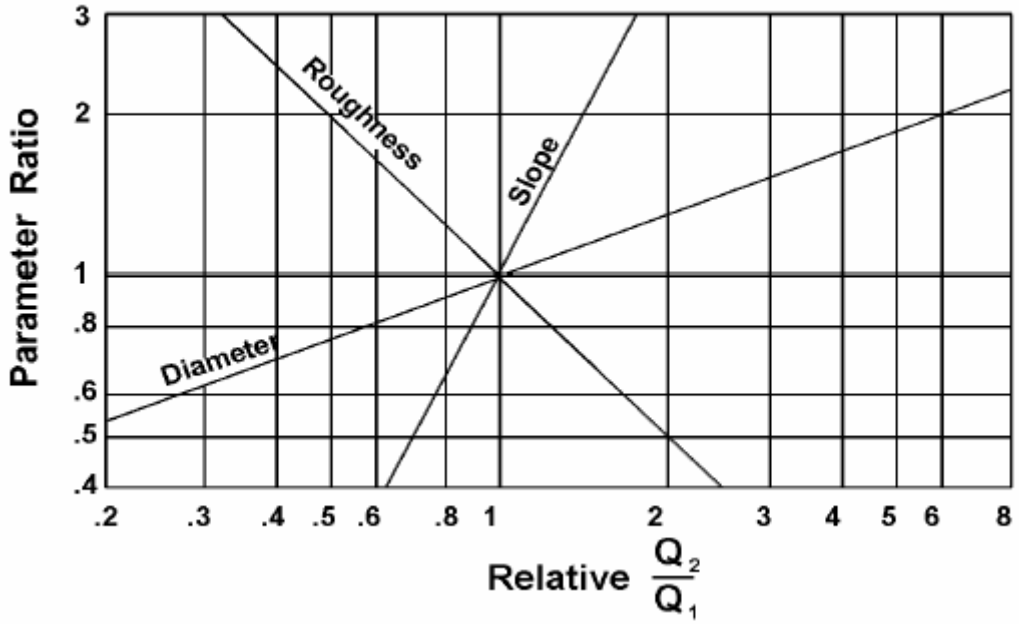


Figure 3.2-17 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

Table 3.2-9 Manning's Coefficients for Storm Drain Conduits
(HEC 22, 2001)

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	Smooth	0.010-0.011
Concrete Boxes	Smooth	0.012-0.015
Spiral Rib Metal Pipe	Smooth	0.012-0.013
Corrugated Metal Pipe, Pipe-Arch and Box (Annular or Helical Corrugations -- see Figure B-3 in Reference 2, Manning's n varies with barrel size)	68 by 13 mm 2-2/3 by 1/2 in Annular	0.022-0.027
	68 by 13 mm 2-2/3 by 1/2 in Helical	0.011-0.023
	150 by 25 mm 6 by 1 in Helical	0.022-0.025
	125 by 25 mm 5 by 1 in	0.025-0.026
	75 by 25 mm 3 by 1 in	0.027-0.028
	150 by 50 mm 6 by 2 in Structural Plate	0.033-0.035
	230 by 64 mm 9 by 2-1/2 in Structural Plate	0.033-0.037
Corrugated Polyethylene	Smooth	0.009-0.015
Corrugated Polyethylene	Corrugated	0.018-0.025
Polyvinyl chloride (PVC)	Smooth	0.009-0.011

*NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

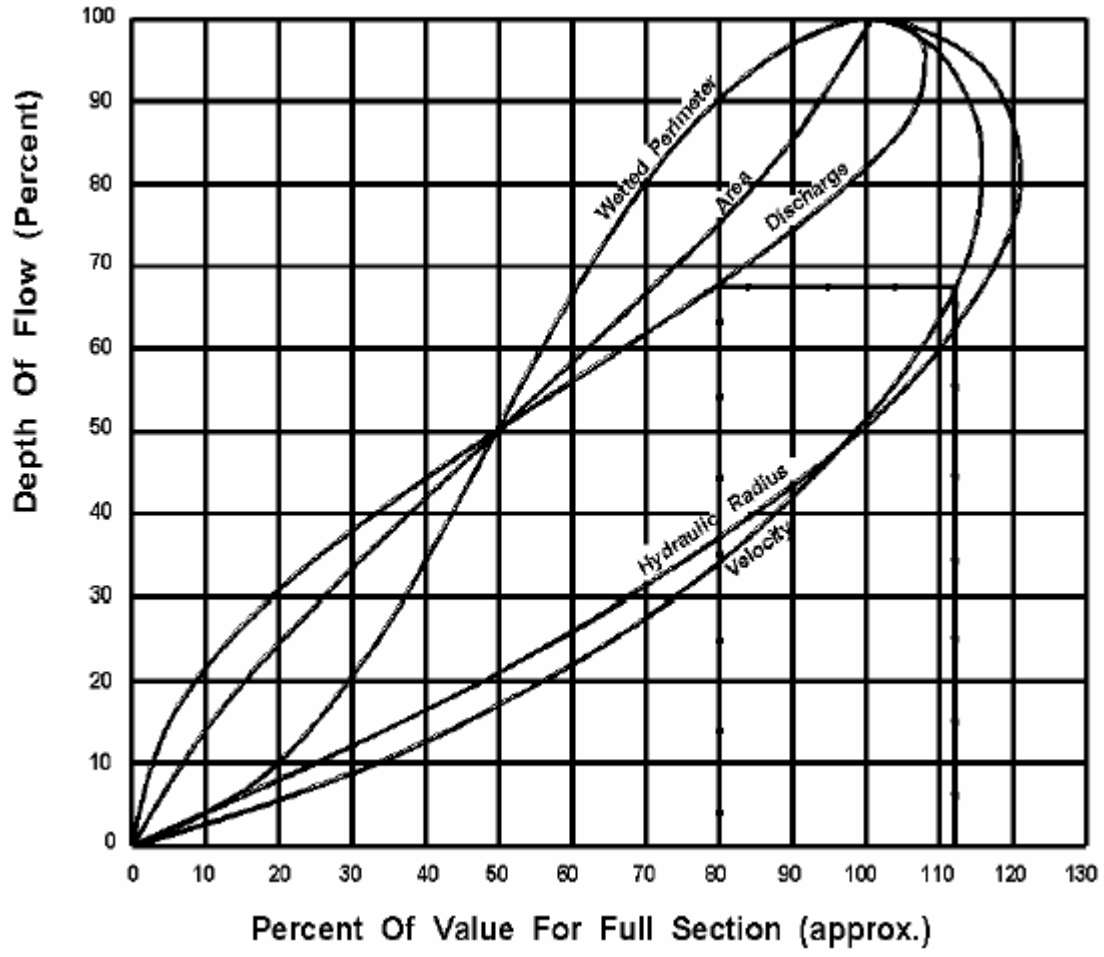


Figure 3.2-18a Hydraulic Elements of Circular Section
(HEC 22, 2001)

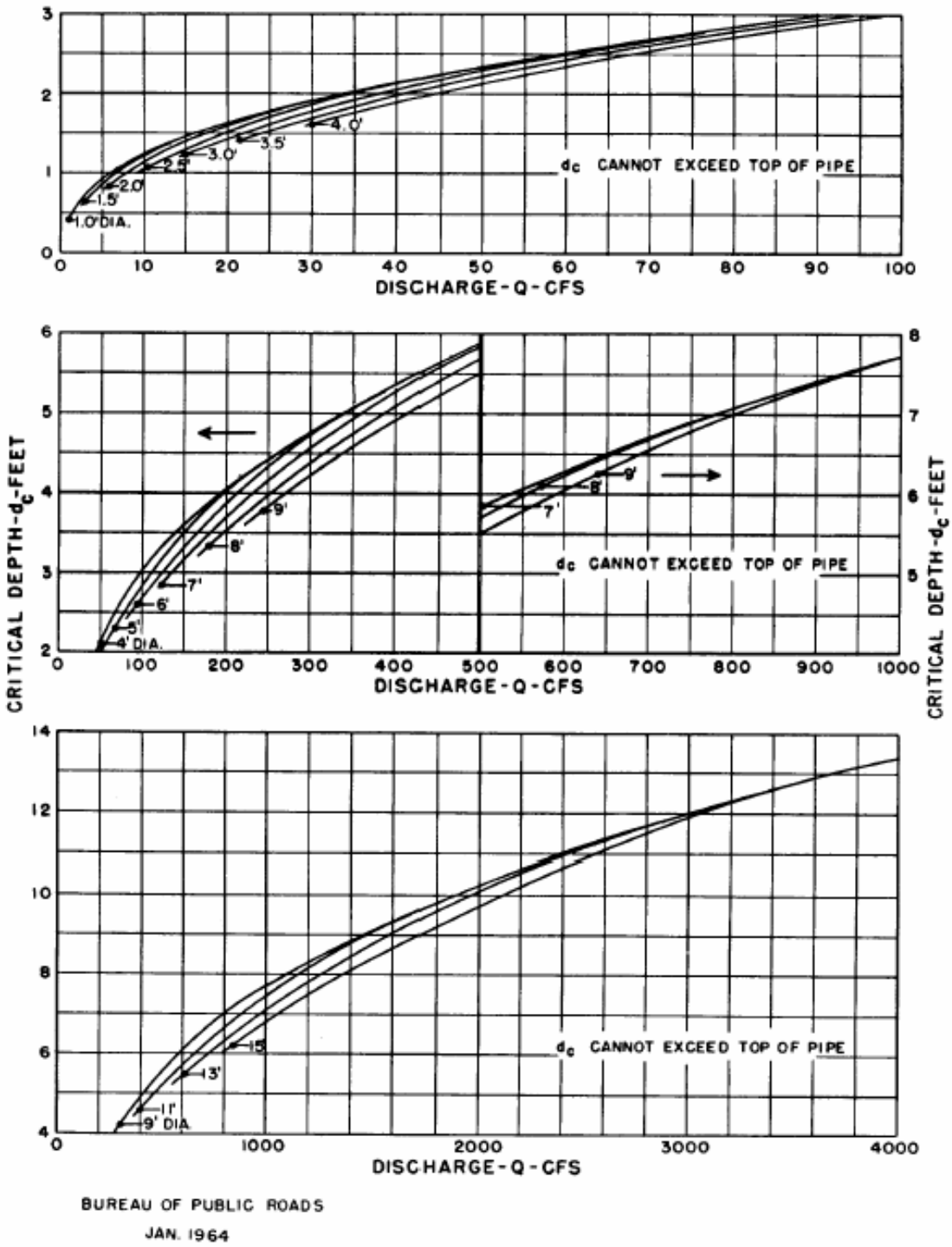


Figure 3.2-18b Critical Depth in Circular Pipe-English Units
(HEC 22, 2001)

3.2.8.7 Minimum Grades and Desirable Velocities

The minimum slopes are calculated by the modified Manning's formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \quad (3.2.22)$$

where:

S = the slope of the hydraulic grade line, ft/ft
n = Manning's roughness coefficient
V = mean velocity of flow, ft/s
R = hydraulic radius, ft (area dived by wetted perimeter)

For circular conduits flowing full but not under pressure, R=D/4, and the hydraulic grade line is equal to the slope of the pipe. For these conditions equation 3.2.22 may be expressed as:

$$S = 2.87(nV)^2/D^{4/3} \quad (3.2.23)$$

For a minimum velocity of 2.5 fps, the minimum slope equation becomes:

$$S = 17.938(n^2/D^{4/3}) \quad (3.2.24)$$

where:

D = diameter, ft

Table 3.2-10 gives minimum slopes for two commonly used materials: concrete pipe with an n-value of 0.013 and corrugated metal pipe with an n-value of 0.024.

Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposits of solid materials; otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity of the sewer. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 3.2-10 gives minimum slopes for two commonly used materials: concrete pipe (n = 0.013) and corrugated metal pipe (n = 0.024), flowing at 2.5 fps.

Desirable Velocities

Velocities in sewers are important mainly because of the possibilities of excessive erosion on the storm drain inverts. Table 3.2-11 shows the desirable velocities for most storm drainage design.

Table 3.2-10 Minimum Grades for Storm Drains for 2.5 fps

Pipe Size (inches)	Concrete Pipe (n = 0.013) Slope ft/ft	Corrugated Metal Pipe (n = 0.024) Slope ft/ft
15	0.0023	0.0077
18	0.0018	0.0060
21	0.0014	0.0049
24	0.0012	0.0041
27	0.0010	0.0035
30	0.0009	0.0030
33	0.0008	0.0027
36	0.0007	0.0024
39	0.0006	0.0021
42	0.0006	0.0020
45	0.0005	0.0018
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0003	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0002	0.0008
96	0.0002	0.0006

Table 3.2-11 Desirable Velocity in Storm Drains

Description	Maximum Desirable Velocity
Culverts (All types)	15 fps.
Storm Drains (Inlet laterals)	No Limit
Storm Drains (Collectors)	15 fps.
Storm Drains (Mains)	12 fps.

3.2.8.8 Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural storm water control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see Section 2.2 for more information).

3.2.8.9 Energy Grade Line/Hydraulic Grade Line

The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head.

$$E = V^2/2g + p/\gamma + z \qquad (3.2.25)$$

where:

E	=	Total energy, ft
$V^2/2g$	=	Velocity head, ft (kinetic energy)
p	=	Pressure, lbs/ft^2
γ	=	Unit weight of water, 62.4 lbs/ft^3
p/γ	=	Pressure head, ft (potential energy)
z	=	Elevation head, ft (potential energy)

Bernoulli's Law expressed between points one (1) and two (2) in a closed conduit accounts for all energy forms and energy losses. The general form of the law may be written as:

$$V_1^2/2g + p_1/\gamma + z_1 = V_2^2/2g + p_2/\gamma + z_2 - H_f - \Sigma H_m \qquad (3.2.26)$$

where:

H_f	=	Pipe friction loss, ft
ΣH_m	=	Sum of minor or form losses, ft

The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. In order to develop the EGL it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses. The friction losses can be calculated using the Manning's Equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. Various tables and calculations exist for developing the value of K depending on the structure being evaluated for loss. Knowledge of the location of the EGL is critical to the understanding and estimating the location of the hydraulic grade line (HGL).

The hydraulic grade line (HGL) is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions.

HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to pipe flow as well as open channel flow. Figure 3.2-19 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes.

When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface. When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, this condition lies in between open channel flow and pressure flow. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

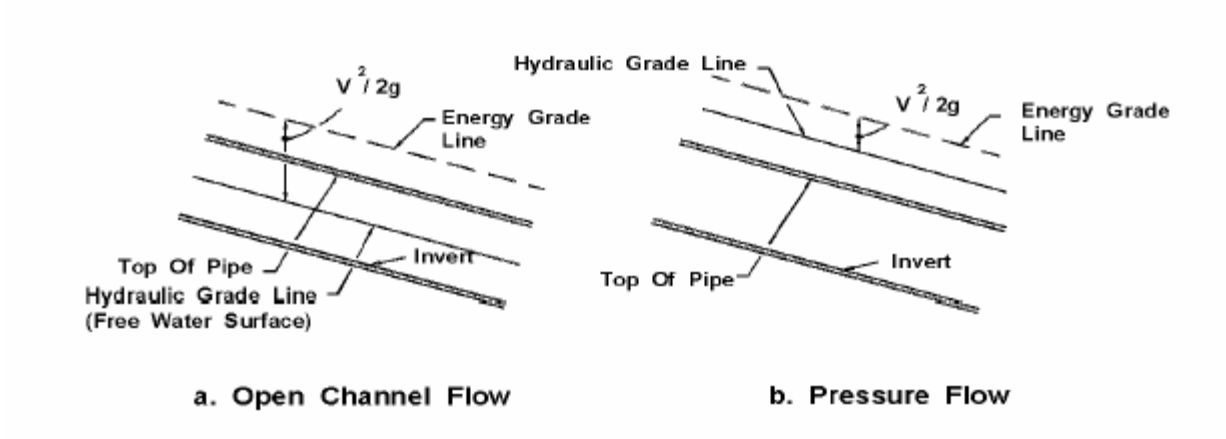


Figure 3.2-19 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

Inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drainage systems can often alternate between pressure and open channel flow conditions from one section to another.

A detailed procedure for evaluating the energy grade line and the hydraulic grade line for storm drainage systems is presented in section 3.2.8.12.

3.2.8.10 Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water. The procedure for calculating the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump or otherwise lift the water to the elevation of the outfall.

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 3.2-8 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and 100-year design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 3.2-8 and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 3.2-12 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio	2-year design		5-year design	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
Area ratio	**10-year design**		**25-year design**	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25

Table 3.2-12 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio	50-year design		100-year design	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pump station.

Energy dissipation may be required to protect the storm drain outlet. Protection is usually required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected. See Section 4.7 for guidance on design of Energy Dissipation Structures.

The **orientation of the outfall** is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

3.2.8.11 Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The following sections present relationships for estimating typical energy losses in storm drainage systems. The application of some of these relationships is included in the design example in section 8.2.8.13.

3.2.8.11.1 Pipe Friction Losses

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows:

$$H_f = S_f L \qquad (3.2.27)$$

where:

H_f = friction loss, ft

S_f = friction slope, ft/ft
L = length of pipe, ft

Section 3.2.8.6 gives the equation for computing the friction loss in pipes flowing full.

The friction slope in equation 3.2.27 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by equation 3.2.27, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow can be determined by the use of Equation 3.2.20.

3.2.8.11.2 Exit Losses

The exit loss from a storm drain outlet is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$H_o = 1.0\ [(V_o^2/2g) - (V_d^2/2g)] \qquad (3.2.28)$$

where:

V_o = average outlet velocity
V_d = channel velocity downstream of outlet

Note that when $V_d = 0$, as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

3.2.8.11.3 Bend Losses

The bend loss coefficient for storm drain design is minor but can be estimated using the following formula (AASHTO, 1991):

$$h_b = 0.0033\ (\Delta)\ (V^2/2g) \qquad (3.2.29)$$

where:

Δ = angle of curvature in degrees

3.2.8.11.4 Transition Losses

A transition is a location where a conduit or channel changes size. Typically, transitions should be avoided and access holes should be used when pipe size increases. However, sometimes transitions are unavoidable. Transitions include expansions, contractions, or both. In small storm drains, transitions may be confined within access holes. However, in larger storm drains or when a specific need arises, transitions may occur within pipe runs as illustrated in Figures 3.2-14 and 3.2-20.

Energy losses in expansions or contractions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends. Contraction and expansion losses can be evaluated with equations 3.2-30 and 3.2-31 respectively.

$$H_c = K_c\ [V_1^2/(2g) - V_2^2/(2g)] \qquad (3.2.30)$$

$$H_e = K_e\ [V_1^2/(2g) - V_2^2/(2g)] \qquad (3.2.31)$$

where:

K_e = expansion coefficient
K_c = contraction coefficient (0.5 K_e)
V_1 = velocity upstream of transition
V_2 = velocity downstream of transition
g = acceleration due to gravity (32.2 ft/s_2)

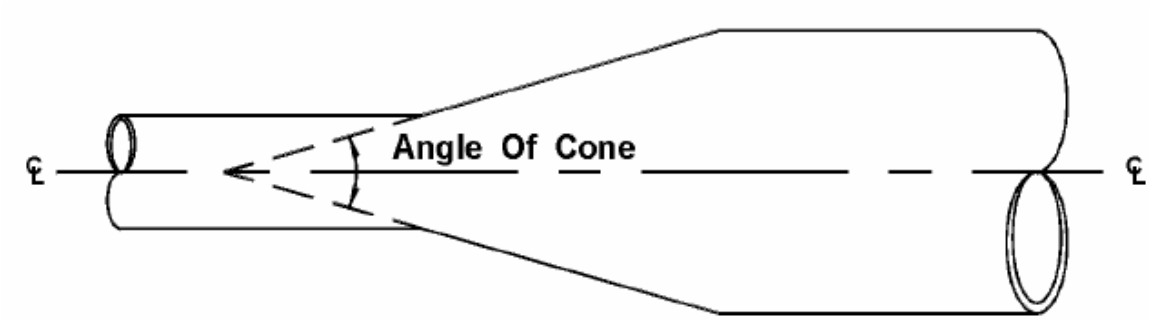


Figure 3.2-20 Angle of Cone for Pipe Diameter Changes

For gradual contractions, it has been observed that K_c = 0.5 K_e. Typical values of K_e for gradual enlargements are tabulated in Table 3.2-10a. Typical values of K_c for sudden contractions are tabulated in Table 3.2-10b. The angle of the cone that forms the transition is defined in Figure 3.2-20.

Table 3.2-13a Typical Values for K_e for Gradual Enlargement of Pipes in Non-Pressure Flow

D_2/D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00
D_2/D_1= Ratio of Diameter of larger pipe to smaller pipe (ASCE, 1992)							

Table 3.2-13b Typical Values of K_c for Sudden Pipe Contractions

D_2/D_1	K_c
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1	0
D_2/D_1= Ratio of Diameter of smaller pipe to larger pipe (ASCE, 1992)	

For storm drain pipes functioning under pressure flow, the loss coefficients listed in Tables 3.2-14 and 3.2-15 can be used with Equation 3.2.32 for sudden and gradual expansions respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 3.2-16 can be used in conjunction with Equation 3.2.33 (ASCE, 1992).

$$H_e = K_e (V_1^2/2g) \quad (3.2.32)$$

$$H_c = K_c (V_2^2/2g) \quad (3.2.33)$$

where:

K_e = expansion coefficient (Tables 3.2-14 and 3.2-15)

K_c = contraction coefficient (Table 3.2.16)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

g = acceleration due to gravity 32.2 ft/s^2

Table 3.2-14 Values of K_e for Determining Loss of Head due to Sudden Enlargement in Pipes

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

Table 3.2-15 Values of K_e for Determining Loss of Head due to Gradual Enlargement in Pipes

D_2/D_1	Angle of Cone										
	2°	6°	10°	15°	20°	25°	30°	35°	40°	50°	60°
1.1	0.01	0.01	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.21	0.23
1.2	0.02	0.02	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.35	0.37
1.4	0.02	0.03	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.50	0.53
1.6	0.03	0.04	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.57	0.61
1.8	0.03	0.04	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.61	0.65
2.0	0.03	0.04	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.63	0.68
2.5	0.03	0.04	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.65	0.70
3.0	0.03	0.04	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.66	0.71
∞	0.03	0.05	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.67	0.72

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
Angle of cone is the angle in degrees between the sides of the tapering section
(ASCE, 1992)

Table 3.2-16 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.11	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31

Table 3.2-16 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

3.2.8.11.5 Junction Losses

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j=[((Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta)) / (0.5g(A_o-A_i))] + h_i - h_o \qquad (3.2.34)$$

where:

H_j = junction loss (ft)

Q_o, Q_i, Q_l = outlet, inlet, and lateral flows respectively (ft^3/s)

V_o, V_i, V_l = outlet, inlet, and lateral velocities, respectively (ft/s)

h_o, h_i = outlet and inlet velocity heads (ft)

A_o, A_i = outlet and inlet cross-sectional areas (ft^2)

θ = angle between the inflow and outflow pipes (Figure 3.2-21)

3.2.8.11.6 Inlet and Access Hole Losses - Preliminary Estimate

The initial layout of a storm drain system begins at the upstream end of the system. The designer must estimate sizes and establish preliminary elevations as the design progresses downstream. An approximate method for estimating losses across an access hole is provided in this section. This is a preliminary estimate only and will not be used when the energy grade line calculations are made. Methods defined in Section 3.2.8.11.7 will be used to calculate the losses across an access hole when the energy grade line is being established.

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 3.2.35. Applicable coefficients (K_{ah}) are tabulated in Table 3.2-17. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. However, this method is used only in the preliminary design process and should not be used in the EGL calculations.

$$H_{ah}=K_{ah} (V_o^2/2g) \qquad (3.2.35)$$

Table 3.2-17 Head Loss Coefficients (FHA, Revised 1993)	
Structure Configuration	**K_{ah}**
Inlet-straight run	0.5
Inlet-angled through	
90°	1.5
60°	1.25
45°	1.1
22.5°	0.7
Manhole-straight run	0.15
Manhole-angled through	
90°	1
60°	0.85
45°	0.75
22.5°	0.45

3.2.8.11.7 Inlet and Access Hole Losses for EGL Calculations - Energy-Loss Methodology

Various methodologies have been advanced for evaluating losses at access holes and other flow junctions. The energy loss method presented in this section is based on laboratory research and does not apply when the inflow pipe invert is above the water level in the access hole.

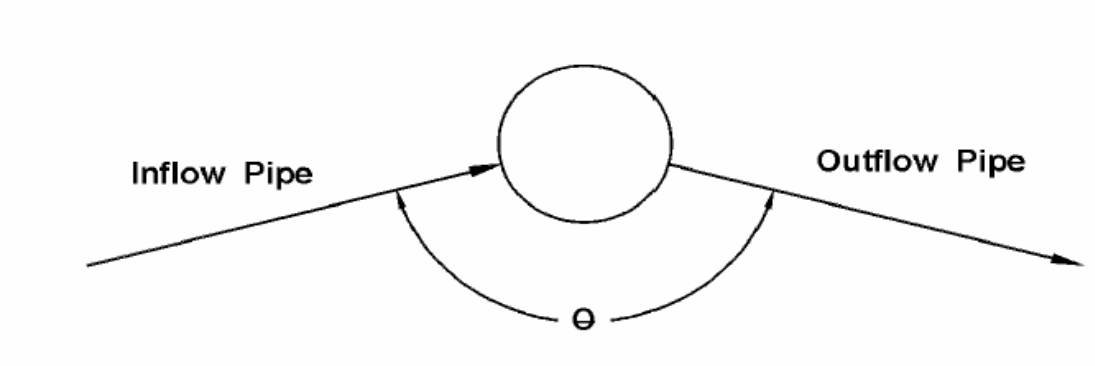


Figure 3.2-21. Head Loss Coefficients

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by Equation 3.2.36. Experimental studies have determined that the K value can be approximated by the relationship in Equation 3.2.37 when the inflow pipe invert is below the water level in the access hole.

$$H_{ah} = K\,(V_o^2/2g) \qquad (3.2.36)$$

$$K = K_O\, C_D\, C_d\, C_Q\, C_p\, C_B \qquad (3.2.37)$$

where:

- K = adjusted loss coefficient
- K_o = initial head loss coefficient based on relative access hole size
- C_D = correction factor for pipe diameter (pressure flow only)
- C_d = correction factor for flow depth

C_Q = correction factor for relative flow
C_p = correction factor for plunging flow
C_B = correction factor for benching
V_o = velocity of outlet pipe

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in *Hydraulic Design of Highway Culverts* (HDS-5, 1985). If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K in Equation 3.2.36 to K_e as reported in Table 3.2-18. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS- 5 (for example see Figure 4.3-2a in Section 4.2).

The initial head loss coefficient, K_o in Equation 3.2.37, is estimated as a function of the **relative access hole** size and the angle of deflection between the inflow and outflow pipes as represented in Equation 3.2.6. This deflection angle is represented in Figure 3.2-21.

$$K_o = 0.1\,(b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta \qquad (3.2.38)$$

where:

θ = angle between the inflow and outflow pipes (Figure 3.2-21)
b = access hole or junction diameter
D_o = outlet pipe diameter

A change in head loss due to differences in **pipe diameter** is only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. In these cases a correction factor for pipe diameter, C_D, is computed using Equation 3.2.39. Otherwise C_D is set equal to 1.

$$C_D=(D_o/D_i)^3 \qquad (3.2.39)$$

where:

D_o = outgoing pipe diameter
D_i = inflowing pipe diameter

Table 3.2-18 Coefficients for Culverts; Outlet Control, Full, or Partly Full

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° levels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Project from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° 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 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/2 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.05
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End-section conforming to fill slope," made of either metal or concrete, 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.
(Source: Reference HDS No.5, 1985)

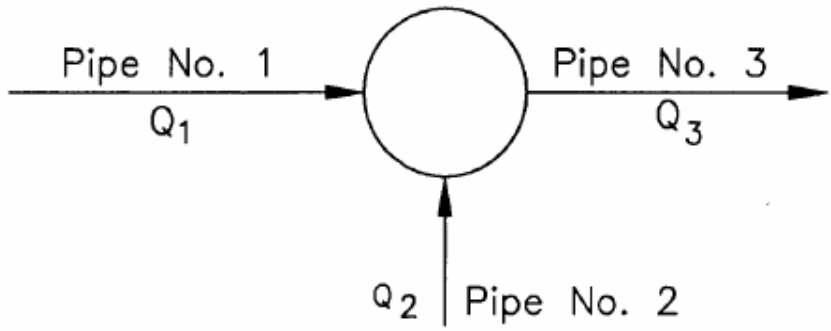


Figure 3.2-22 Relative flow effect

The correction factor for **flow depth**, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho}/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to 1. To determine the applicability of this factor, the water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor is calculated using Equation 3.2.38.

$$C_D = 0.5(d_{aho}/D_o)^{0.6} \qquad (3.2.40)$$

where:

d_{aho} = water depth in access hole above the outlet pipe invert

D_o = outlet pipe diameter

The correction factor for **relative flow**, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed using Equation 3.2.39. The correction factor is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0.

$$C_Q = (1 - 2\sin\theta)\,[1-(Q_i / Q_o)]^{0.75} + 1 \qquad (3.2.41)$$

where:

C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (Figure 3.2-22)

Q_i = flow in the inflow pipe

Q_o = flow in the outflow pipe

As can be seen from Equation 3.2.39, C_Q is a function of the angle of the incoming flow as well as the ratio of inflow coming through the pipe of interest and the total flow out of the structure. To illustrate this effect, consider the access hole shown in Figure 3.2-23 and assume the following two cases to determine the correction factor of pipe number 2 entering the access hole. For each of the two cases, the angle between the inflow pipe number 1 and the outflow pipe, θ, is 180°.

Case 1:
$Q_1 = 3\ ft^3/s$
$Q_2 = 1\ ft^3/s$
$Q_3 = 4\ ft^3/s$

Using Equation 3.2.39,
$C_Q = (1 - 2 \sin \theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 3/4)^{0.75} + 1$
$C_Q = 1.35$

Case 2:
$Q_1 = 1.0\ ft^3/s$
$Q_2 = 3.0\ ft^3/s$
$Q_3 = 4.0\ ft^3/s$

Using Equation 3.2.39,
$C_Q = (1 - 2 \sin \theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 1/4)^{0.75} + 1$
$C_Q = 1.81$

The correction factor for **plunging flow**, C_p, is calculated using Equation 3.2.40. This correction factor corresponds to the effect another inflow pipe, plunging into the access hole, has on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 3.2-23, C_p is calculated for pipe #1 when pipe #2 discharges plunging flow. The correction factor is only applied when $h > d_{aho}$. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow in the bottom of the access hole. Otherwise, the value of C_p is equal to 1.0. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses would be computed using Equation 3.2.40.

$$C_p = 1 + 0.2(h/D_o)\,[(h - d_{aho})/D_o] \qquad (3.2.42)$$

where:

C_p = correction for plunging flow

h = vertical distance of plunging flow from the flow line of the higher elevation inlet pipe to the center of the outflow pipe

D_o = outlet pipe diameter

d_{aho} = water depth in access hole relative to the outlet pipe invert

The correction for **benching** in the access hole, C_B, is obtained from Table 3.2-15. Figure 3.2-24 illustrates benching methods listed in Table 3.2-15. Benching tends to direct flow through the access hole, resulting in a reduction in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 3.2-19 Correction for Benching
(HEC 22, 2001)

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

*pressure flow, $d_{aho}/D_o \geq 3.2$

**free surface flow, $d_{aho}/D_o \leq 3.2$

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe using the energy-loss method, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

3.2.8.11.8 Composite Energy Loss Method

The Energy Loss Method described in Section 3.2.8.11.7 resulted from preliminary experimental and analytical techniques that focused on relatively simple access hole layout and a small number of inflow pipes. A more suitable method is available to analyze complex access holes that have, for example, many inflow pipes. This complex method, referred to as the Composite Energy Loss Method, is implemented in the FHWA storm drain analysis and design package HYDRA (GKY, 1994). Details on the method are described in the HYDRA program technical documentation and the associated research report (Chang, et. al., 1994).

This complex minor loss computation approach focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe (Chang, et. al., 1994). The methodology can be applied by determining the estimated energy loss through an access hole given a set of physical and hydraulic parameters. Computation of the energy loss allows determination and analysis of the energy gradeline and hydraulic gradeline in pipes upstream of the access hole. This methodology only applies to subcritical flow in pipes.

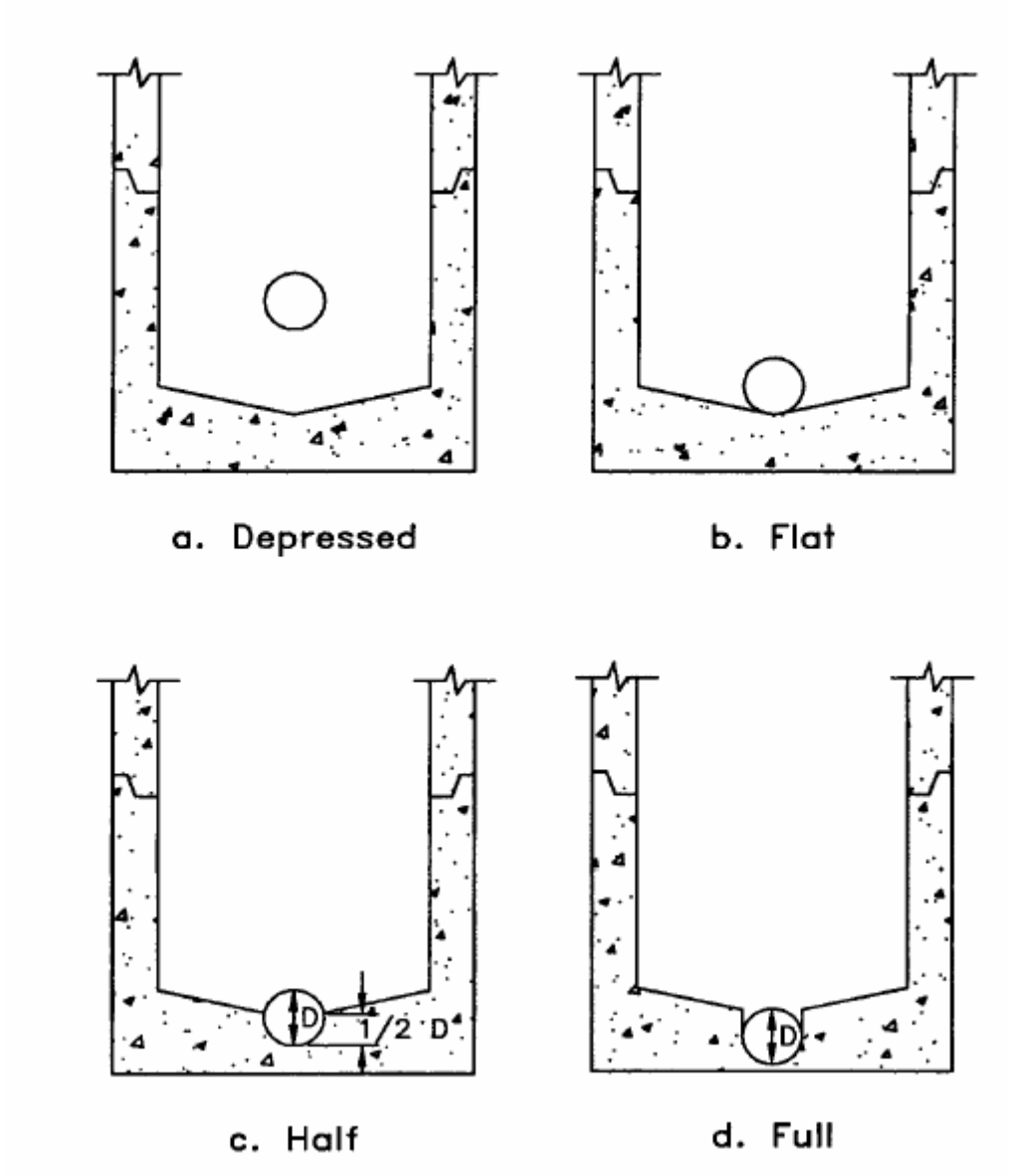


Figure 3.2-23 Access to Benching Methods

3.2.8.12 Preliminary Design Procedure

The preliminary design of storm drains can be accomplished by using the following steps and the storm drain computation sheet provided in Figure 3.2-25. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed (See Section 3.2.8.9).

Step 1 Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

a. Location of storm drains.
b. Direction of flow.

c. Location of access holes and other structures.
d. Number or label assigned to each structure.
e. Location of all existing utilities (water, sewer, gas, underground cables, etc.).

Step 2 Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system:

a. Drainage areas.
b. Runoff coefficients.
c. Travel time

Step 3 Using the information generated in Steps 1 and 2, complete the following information on the design form for each run of pipe starting with the upstream most storm drain run:

a. "From" and "To" stations, Columns 1 and 2b, "Length" of run, Column 3
b. "Length" of run, Column 3
c. "Inc." drainage area, Column 4

   The incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.
d. "C," Column 6

   The runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases a composite runoff coefficient will need to be computed.
e. "Inlet" time of concentration, Column 9

   The time required for water to travel from the hydraulically most distant point of the drainage area to the inlet at the upstream end of the storm drain run under consideration.
f. "System" time of concentration, Column 10

   The time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain run under consideration. For the upstream most storm drain run this value will be the same as the value in Column 9. For all other pipe runs this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous run together to get the system time of concentration at the upstream end of the section under consideration (See Section 3.2.8.3 for a general discussion of times of concentration).

Step 4 Using the information from Step 3, compute the following:

a. "TOTAL" area, Column 5

   Add the incremental area in Column 4 to the previous sections total area and place this value in Column 5.
b. "INC." area x "C," Column 7

   Multiply the drainage area in Column 4 by the runoff coefficient in Column 6. Put the product, CA, in Column 7.
c. "TOTAL" area x "C," Column 8

   Add the value in Column 7 to the value in Column 8 for the previous storm drain run and put this value in Column 8.
d. "I," Column 11

   Using the larger of the two times of concentration in Columns 9 and 10, and an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity, I, and place this value in Column 11.

e. "TOTAL Q," Column 12

Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.

f. "SLOPE," Column 21

Place the pipe slope value in Column 21. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.

g. "PIPE DIA.," Column 13

Size the pipe using relationships and charts presented in Section 3.2.8.6 to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to a full gravity flow. Since most calculated sizes will not be available, a nominal size will be used. The designer will decide whether to go to the next larger size and have part full flow or whether to go to the next smaller size and have pressure flow.

h. "CAPACITY FULL," Column 14

Compute the full flow capacity of the selected pipe using Equation 3.2.18 and put this information in Column 14.

i. "VELOCITIES," Columns 15 and 16

Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from V = Q/A, Equations 3.2.17 and 3.2.18. If the pipe is not flowing full, the velocity can be determined from Figure 3.2-18a.

j. "SECTION TIME," Column 17

Calculate the travel time in the pipe section by dividing the pipe length (Column 3) by the design flow velocity (Column 16). Place this value in Column 17.

k. "CROWN DROP," Column 20

Calculate an approximate crown drop at the structure to off-set potential structure energy losses using Equation 3.2.33 introduced in Section 3.2.8.11.6. Place this value in Column 20.

l. "INVERT ELEV.," Columns 18 and 19

Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this section of pipe, including any pipe size changes that occurred along the section.

Step 5 Repeat steps 3 and 4 for all pipe runs to the storm drain outlet. Use equations and nomographs to accomplish the design effort.

Step 6 Check the design by calculating the energy grade line and hydraulic grade line as described in Section 3.2.8.9.

PRELIMINARY STORM DRAIN COMPUTATION SHEET

COMPUTED BY _____ DATE _____
CHECKED BY _____ DATE _____
PAGE _____ OF _____

ROUTE _____
SECTION _____
COUNTY _____

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	( )	INC ( )	TOTAL ( )		INC. ( )	TOTAL ( )	INLET (min)	SYSTEM (min)	(/hr)	(^{3} /s)	( )	(^{3}/s)	FULL (/s)	DESIGN (/s)	(min)	U/S ( )	D/S ( )	( )	(/)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 3.2-24 Preliminary Storm Drain Computation Sheet

3.2.8.13 Energy Grade Line Evaluation Procedure

This section presents a step-by-step procedure for manual calculation of the energy grade line (EGL) and the hydraulic grade line (HGL) using the energy loss method. For most storm drainage systems, computer methods such as HYDRA (GKY, 1994) are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process so that he can better interpret the output from computer generated storm drain designs.

Figure 3.2-25 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figure 3.2-26 and Figure 3.2-27 can be used to document the procedure outlined below.

Before outlining the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity). Table A (Figure 3.2-26) is used to calculate the HGL and EGL elevations while table B (Figure 3.2-27) is used to calculate the pipe losses and structure losses. Values obtained in table B are transferred to table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

EGL computations begin at the outfall and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

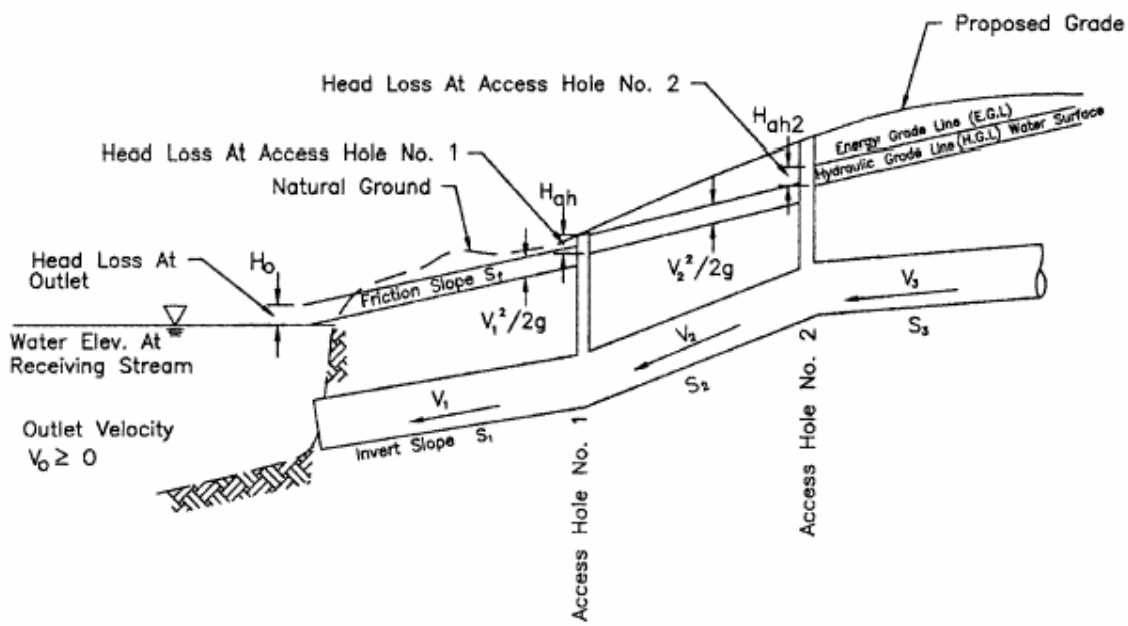


Figure 3.2-25 Energy and Hydraulic Grade Line Illustration

The EGL computational procedure follows:

Step 1 The first line of Table A includes information on the system beyond the end of the conduit system. Define this as the stream, pool, existing system, etc. in column 1A. Determine the EGL and HGL for the downstream receiving system. If this is a natural body of water, the HGL will be at the water surface. The EGL will also be at the water surface if no velocity is assumed or will be a velocity head above the HGL if there is a velocity in the water body. If the new system is being connected to an existing storm drain system, the EGL and the HGL will be that of the receiving system. Enter the HGL in Column 14A and the EGL in Column 10A of the first line on the computation sheet.

Step 2 Identify the structure number at the outfall (this may be just the end of the conduit, but it needs a structure number), the top of conduit (TOC) elevation at the outfall end, and the surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 15A, and 16A respectively. Also add the structure number in Col.1B.

Step 3 Determine the EGL just upstream of the structure identified in Step 2. Several different cases exist as defined below when the conduit is flowing full:

Case 1: If the TW at the conduit outlet is greater than $(d_c + D)/2$, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions.

Case 2: If the TW at the conduit outlet is less than $(d_c + D)/2$, the EGL will be the HGL plus the velocity head for the conduit flow conditions. The equivalent hydraulic grade line, EHGL, will be the invert plus $(d_c + D)/2$.

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A

COMPUTED BY _____ DATE _____ ROUTE _____
CHECKED BY _____ DATE _____ SECTION _____
PAGE _____ OF _____ COUNTY _____
INITIAL TAILWATER ELEV. _____

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K (table B)	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	( )	(^{3}/s)	( )	(/s)	( )	( )	( )	(/)	( )	( )		( )	( )	( )	( )	( )
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)

Figure 3.2-26 Energy Grade Line Computation Sheet - Table A

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B

COMPUTED BY _____ DATE _____ ROUTE _____
CHECKED BY _____ DATE _____ SECTION _____
PAGE _____ OF _____ COUNTY _____

Str. ID	Pipe Losses ( )						Structure Losses ( )							
	H_f	h_b	H_c	H_e	H_j	Total	d_{aho}	K_o	C_D	C_d	C_Q	C_P	C_B	K
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)

Figure 3.2-27 Energy Grade Line Computation Sheet - Table B

The velocity head needed in either Case 1 or 2 will be calculated in the next steps, so it may be helpful to complete Step 4 and work Step 5 to the point where velocity head (Col. 7A) is determined and then come back and finish this step. Put the EGL in Column 13A.

Note: The values for d_c for circular pipes can be determined from Figure 3.2-18b. Charts for other conduits or other geometric shapes can be found in *Hydraulic Design of Highway Culverts*, HDS-5, and cannot be greater than the height of the conduit.

Step 4 Identify the structure ID for the junction immediately upstream of the outflow conduit (for the first conduit) or immediately upstream of the last structure (if working with subsequent lines) and enter this value in Columns 1A and 1B of the next line on the computation sheets. Enter the conduit diameter (D) in column 2A, the design discharge (Q) in Column 3A, and the conduit length (L) in Column 4A.

Step 5 If the barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 7A. Put "full" in Column 6a and not applicable (n/a) in Column 6b of Table A. Continue with Step 6. If the barrel flows only partially full, continue with Step 5A.

Note: If the pipe is flowing full because of high tailwater or because the pipe has reached its capacity for the existing conditions, the velocity will be computed based on continuity using the design flow and the full cross sectional area. Do not use the full flow velocity determined in Column 15 of the Preliminary Storm Drain Computation Form for part-full flow conditions. For part-full conditions discussed in Step 5, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL/HGL calculations.

Step 5A Part full flow: Using the hydraulic elements graph in Figure 3.2-18a with the ratio of part full to full flow (values from the preliminary storm drain computation form), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5 respectively of Table A. Compute the velocity head ($V_2/2g$) and place in Column 7A.

Step 5B Compute critical depth for the conduit using Figure 3.2-18b. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 6b of Table A.

Step 5C Compare the flow depth in Column 6a (Table A) with the critical depth in Column 6b (Table A) to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 6. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5D. In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for super critical flow in the upstream section of pipe, assure that the EGL is higher in the pipe than in the structure.

Step 5D Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.

Step 5E Enter the structure ID for the next upstream structure on the next line in Columns 1A and 1B. Enter the pipe diameter (D), discharge (Q), and conduit length (L) in Columns 2A, 3A, and 4A respectively of the same line.

Note: After a downstream pipe has been determined to flow in supercritical flow, it is necessary to check each succeeding upstream pipe for the type of flow that exists. This is done by calculating normal depth and critical depth for each pipe. If normal depth is less than the diameter of the pipe, the flow will be open channel flow and the critical depth calculation can be used to determine whether the flow is sub or supercritical. If the flow line elevation through an access hole drops enough that the invert of the upstream pipe is not inundated by the flow in the downstream pipe, the designer goes

back to Step1A and begins a new design as if the downstream section did not exist.

Step 5F Compute normal depth for the conduit using Figure 3.2-18a and critical depth using Figure 3.2-18b. If the conduit is not circular see HDS-5 for additional charts. Enter these values in Columns 6A and 6b of Table A.

Step 5G If the pipe barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Go to Step 3, Case 2 to determine the EGL at the outlet end of the pipe. Put this value in Column 10A and go to Step 6. For part full flow, continue with Step 5H.

Step 5H Part full flow: Compute the velocity of flow in the conduit and enter this value in Column 5A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

Step 5I Compare the flow depth in Column 6a with the critical depth in Column 6b to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 5J. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5K.

Step 5J Subcritical flow upstream: Compute EGL_o at the outlet of the previous structure as the outlet invert plus the sum of the outlet pipe flow depth and the velocity head. Place this value in Column 10A of the appropriate structure and go to Step 9.

Step 5K Supercritical flow upstream: Access hole losses do not apply when the flow in two (2) successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 15B of the intermediate structure (previous line). The HGL at the structure is equal to the pipe invert elevation plus the flow depth. Check the invert elevations and the flow depths both upstream and downstream of the structure to determine where the highest HGL exists. The highest value should be placed in Column 14A of the previous structure line. Perform Steps 20 and 21 and then repeat Steps 5E through 5K until the flow regime returns to subcritical. If the next upstream structure is end-of-line, skip to step 10b then perform Steps 20, 21, and 24.

Step 6 Compute the friction slope (S_f) for the pipe using Equation 3.2.19 divided by L[$S_f = H_f/L = [185\ n^2\ (V^2/2g)]/D^{4/3}$] for a pipe flowing full. Enter this value in Column 8A of the current line. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 7 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_b), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) using Equations 3.2.27 through 3.2.32 and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Column 7B and 9A.

Step 8 Compute the energy grade line value at the outlet of the structure (EGL_o) as the EGL_i elevation from the previous structure (Column 13A) plus the total pipe losses (Column 9A). Enter the EGL_o in Column 10A.

Step 9 Estimate the depth of water in the access hole (estimated as the depth from the outlet pipe invert to the hydraulic grade line in the pipe at the outlet). Computed as EGL_o (Column 10A) minus the pipe velocity head in Column 7A minus the pipe invert elevation (from the preliminary storm drain computation form). Enter this value in Column 8B. If supercritical flow exists in this structure, leave this value blank and skip to Step 5E.

Step 10 If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equations 3.2.36 and 3.2.37. Start by computing the initial structure

head loss coefficient, K_o, based on relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5 as follows:

a. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in Equation 3.2.35 to K_e as reported in Table 3.2-16. Enter this value in Column 15B and 11A, continue with Step 17. Add a note on Table A indicating that this is a drop structure.

b. If the outflow pipe functions under inlet control, compute the depth in the access hole (HGL) using Chart 28 or 29. If the storm conduit shape is other than circular, select the appropriate inlet control nomograph from HDS-5. Add these values to the access hole invert to determine the HGL. Since the velocity in the access hole is negligible, the EGL and HGL are the same. Enter HGL in Col.14A and EGL in Col.13A. Add a note on Table A indicating that this is a drop structure. Go to Step 20.

Step 11 Using Equation 3.2.39 compute the correction factor for pipe diameter, C_D, and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

Step 12 Using Equation 3.2.40 compute the correction factor for flow depth, C_D, and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

Step 13 Using Equation 3.2.41, compute the correction factor for relative flow, C_Q, and enter this value in Column 12B. This factor = 1.0 if there are less than 3 pipes at the structure.

Step 14 Using Equation 3.2.42, compute the correction factor for plunging flow, C_p, and enter this value in Column 13B. This factor = 1.0 if there is no plunging flow. This correction factor is only applied when $h>d_{aho}$.

Step 15 Enter in Column 14B the correction factor for benching, C_B, as determined from Table 3.2-18. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 3.2.37, compute the value of K and enter this value in Column 15B and 11A.

Step 17 Compute the total access hole loss, H_{ah}, by multiplying the K value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

Step 18 Compute EGL_i at the structure by adding the structure losses in Column 12A to the EGL_o value in Column 10A. Enter this value in Column 13A.

Step 19 Compute the hydraulic grade line (HGL) at the structure by subtracting the velocity head in Column 7A from the EGL_i value in Column 13A. Enter this value in Column 14A.

Step 20 Determine the top of conduit (TOC) value for the inflow pipe (using information from the storm drain computation sheet) and enter this value in Column 15A.

Step 21 Enter the ground surface, top of grate elevation or other high water limits at the structure in Column 16A. If the HGL value in Column 14A exceeds the limiting elevation, design modifications will be required.

Step 22 Enter the structure ID for the next upstream structure in Column 1A and 1B of the next line. When starting a new branch line, skip to Step 24.

Step 23 Continue to determine the EGL through the system by repeating Steps 4 through 23. (Begin with Step 2 if working with a drop structure. This begins the design process again as if there were no system down stream from the drop structure).

Step 24 When starting a new branch line, enter the structure ID for the branch structure in Column 1A and 1B of a new line. Transfer the values from Columns 2A through 10A and 2B to 7B associated with this structure on the main branch run to the corresponding columns for the branch line. If flow in the main storm drain at the branch point is subcritical, continue with Step 9; if supercritical, continue with Step 5E.

3.2.8.14 Storm Drain Design Example

The following storm drain design example illustrates the application of the design procedures outlined in Sections 3.2.8.11 and 3.2.8.12.

Example of Preliminary Storm Drain Design

Given: The roadway plan and section illustrated in Figure 3.2-29, duration intensity information in Table 3.2-21, and inlet drainage area information in Table 3.2-20. All grates are type P 50 x 100, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter = 18 in for maintenance purposes.

Find:

(1) Using the procedures outlined in Section 3.2.8.11 determine appropriate pipe sizes and inverts for the system illustrated in Figure 3.2-29.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in Section 3.2.8.12.

Solution:

(1) Preliminary Storm Drain Design

Step 1. Figure 3.2-29 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 3.2-30 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2. Drainage areas, runoff coefficients, and times of concentration are tabulated in Figure 3.2-31. Example problems documenting the computation of these values are included in this chapter.

Starting at the upstream end of a conduit run, Steps 3 and 4 from Section 3.2.8.11 are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 3.2-31. The column by column computations for each section of conduit follow:

Table 3.2-20 Drainage Area Information for Design Example

Inlet No.	Drainage Area (ac)	"C"	Time of Concentration (min)
40	0.64	0.73	3
41	0.35	0.73	2
42	0.32	0.73	2
43	--	--	--
44			

Table 3.2-21 Intensity/Duration Data Design Example									
Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/hr)	7.1	5.9	5.1	4.5	3.5	3	2.6	2.4	1.4

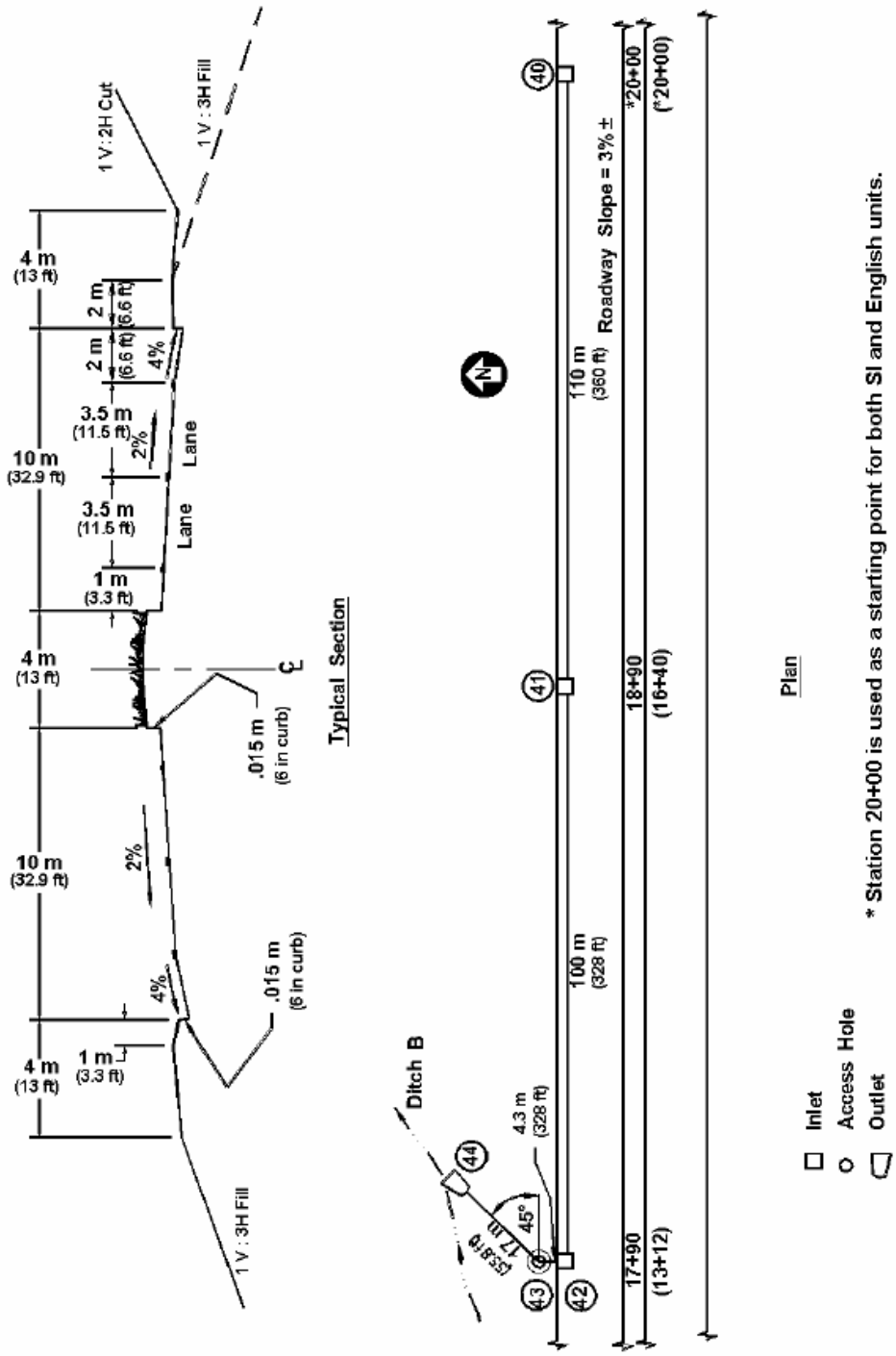


Figure 3.2-28 Roadway Plan and Sections for Example

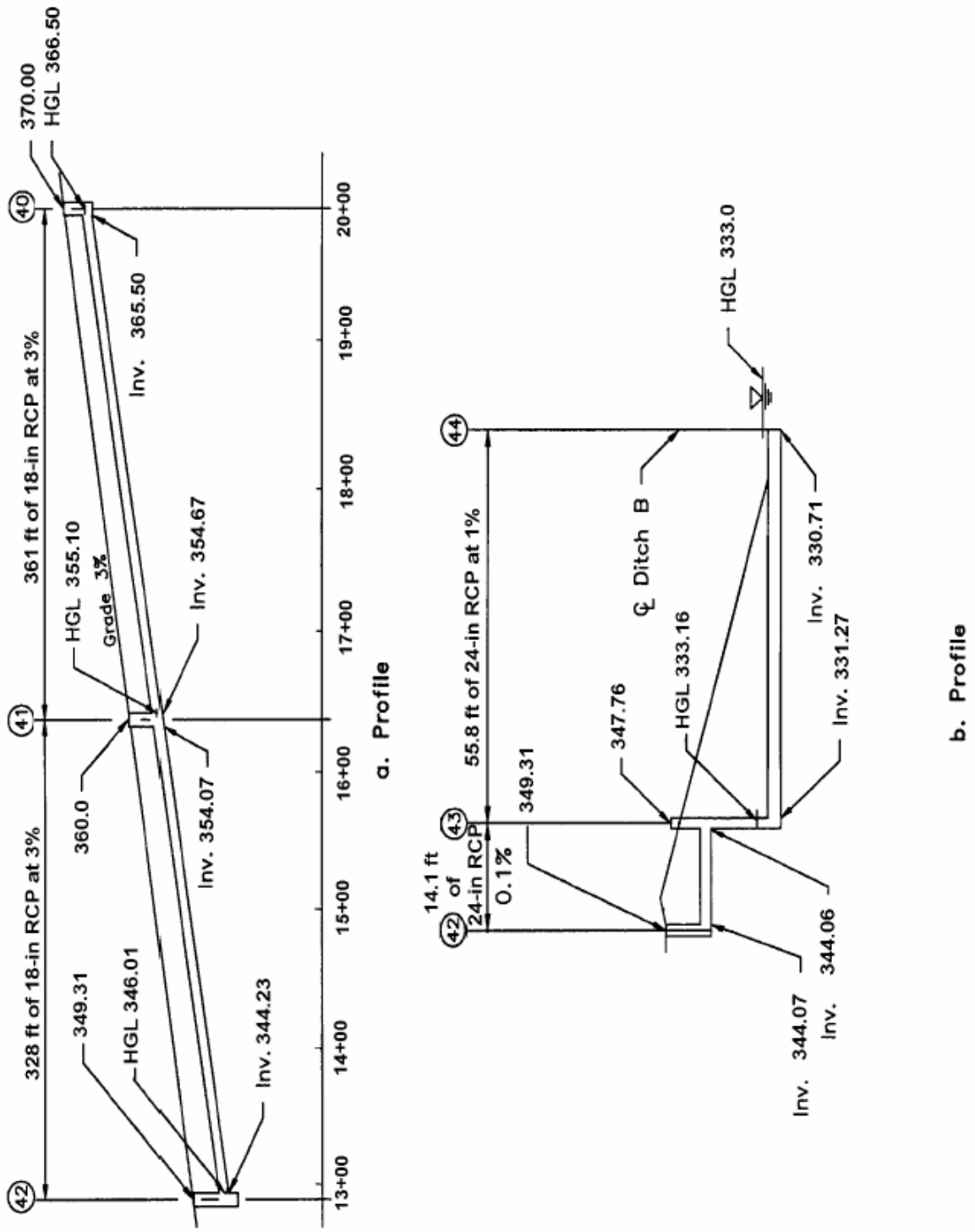


Figure 3.2-29 Storm Drain Profiles for Example

PRELIMINARY STORM DRAIN COMPUTATION SHEET
(English Solution)

COMPUTED BY _____ DATE _____
CHECKED BY _____ DATE _____
PAGE _____ OF _____

ROUTE _____
SECTION _____
COUNTY _____

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	(Ft)	INC (ac)	TOTAL (ac)		INC. (ac)	TOTAL (ac)	INLET (min)	SYSTEM (min)	(in/hr)	(ft^3 /s)	(ft)	(ft^3/s)	FULL (ft /s)	DESIGN (ft /s)	(min)	U/S (ft)	D/S (ft)	(ft)	(ft/ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
40	41	361.0	0.64	0.64	0.73	0.47	0.47	3	5	7.1	3.3	1.50	18.1	10.3	7.52	1	365.50	354.67	----	0.03
41	42	328.0	0.35	0.99	0.73	0.25	0.72	2	5	7.1	5.1	1.5	18.1	10.3	8.7	1	354.07	344.23	0.6	0.03
42	43	14.1	0.32	1.31	0.73	0.23	0.95	2	5	7.1	6.75	2.0	7.12	2.28	2.6	0	344.07	344.06	0.16	0.001
43	44	55.8	0.00	1.31	-	-	0.95	-	5	7.1	6.75	2.0	22.52	7.22	6.10	0	331.27	330.71	12.79	0.01

Figure 3.2-30 Storm Drain Computation Sheet for Example

Structure 40 to 41

Col. 1 From structure 40

Col. 2 To structure 41

Col. 3 Run Length	L = 2000 ft - 1639 ft L = 361 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.64 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.64 ac	Total area up to inlet 40
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.64)(0.73) CA = 0.47 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.47 + 0 ΣCA= 0.47 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 3 min	Figure 3.2-31
Col. 10 Sys. Time	t_c = 3 min (use 5 min)	same as Col. 9 for upstream most section
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21; System time less than 5 minutes therefore, use 5 minutes
Col. 12 Runoff	$Q = C_f (CA) (I)$ Q = (0.47) (7.1) / 1.0 $Q = 3.3\ ft^3/sec$	Equation 2.1.3; C_f = 1.0 Col. 8 times Col. 11 multiplied by 1.0
Col. 21 Slope	S = 0.03	select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(3.3)(0.013)/(0.46)(0.03)^{0.5})]^{0.375}$ D = 0.8 ft D_{min} = 1.5 ft	Equation 3.2.18 or Figure 3.2-16 use D_{min}
Col. 14 Full Cap	$Q_f = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)\ (1.5)^{2.67}\ (0.03)^{0.5}$ $Q_f = 18.1 ft^3/s$	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)\ (1.5)^{0.67}\ (0.03)^{0.5}$ V_f = 10.3 ft/s	Equation 3.2.17 or Figure 3.2-16
Col. 16 Vel. Design	Q/Q_f = 3.3/18.1 = 0.18 V/V_f = 0.73 V = (0.73) (10.3) V = 7.52 ft/s	 Figure 3.2-18a
Col. 17 Sect. Time	t_s = L/V = 361 / 7.52 / 60 t_s = 0.8 min; use 1 min	Col. 3 divided by Col. 16

Col. 20 Crown Drop	= 0	Upstream most invert
Col. 18 U/S Invert	= Grnd - 3.0 ft - dia = 370.0 - 3.0 - 1.5 = 365.5 ft	3 ft = min cover Ground elevation from Figure 3.2-30
Col. 19 D/S Invert	= (365.5) - (361.0)(0.03) = 354.67 ft	Col. 18 - (Col. 3)(Col. 21)

At this point, the pipe should be checked to determine if it still has adequate cover.

354.67 + 1.5 + 3.0 = 359.17 Invert elev. + Diam + min cover
Ground elevation of 360.0 ft is greater than 359.17 ft so OK

Structure 41 to 42

Col. 1 From	= 41	
Col. 2 To	= 42	
Col. 3 Run Length	L = 1639 - 1311 \ L = 328 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.35 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.35 + 0.64 A_t = 0.99 ac	
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.35)(0.73) CA = 0.25 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.25 + 0.47 ΣCA = 0.72 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 2 min	Table 3.2-20
Col. 10 Sys. Time	t_c = 4 min (use 5 min)	Col. 9 + Col. 17 for line 40-41
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21; system time equals 5 min
Col. 12 Runoff	Q = (CA)(I)/(K_u) Q = (0.72) (7.1) / 1.0 Q = 5.1 ft^3/sec by 1.0	Equation 2.1.3 Col. 8 times Col. 11 divided
Col. 21 Slope	S = 0.03	select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(5.1)(0.013)/(0.46)(0.03)^{0.5}]^{0.375}$ D = 0.93 ft D_{min} = 1.5 ft use D_{min}	Equation 3.2.18 or Figure 3.2-16 use D_{min}

Col. 14 Full Cap.	$Q_f = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5}$ $Q_f = 18.1$ ft3/s	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)(1.5)^{0.67}\ (0.03)^{0.5}$ $V_f = 10.3$ ft/s	Equation 3.2.18 or Figure 3.2-16
Col. 16 Vel. Design	$Q/Q_f = 5.1/18.1 = 0.28$ $V/V_f = 0.84$ $V = (0.84)\ (10.3)$ $V = 8.7$ ft/s	Figure 3.2-18a
Col. 17 Sect. Time	$T_s = L/V = 328 / 8.75 / 60$ $T_s = 0.6$ min; use 1 min	Col. 3 divided by Col. 16
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (0.5)(8.7)^2 / [(2)(32.2)]$ $= 0.6$ ft	Equation 3.2.36 with Table 3.2-16 $K_{ah} = 0.5$ for inlet - straight run
Col. 18 U/S Invert	= 354.67 - 0.6 = 354.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= (354.07) - (328)(0.03) = 344.23 ft	Col. 18 - (Col. 3)(Col. 21)

Structure 42 to 43

Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	L = 14.1 ft	Figure 3.2-30
Col. 4 Inlet Area	$A_i = 0.32$ ac	Figure 3.2-31
Col. 5 Total Area	$A_t = 0.32 + 0.99$ $A_t = 1.31$ ac	Col. 4 plus structure 41 total area
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.32)(0.73) CA = 0.23 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.23 + 0.72 ΣCA = 0.95 ac	Col. 7 plus structure 41 total CA values
Col. 9 Inlet Time	$t_i = 2$ min	Table 3.2-20
Col. 10 Sys. Time	$t_c = 5$ min	Col. 9 + Col. 17 for line 40-41 plus Col.17 for line 41-42

Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21
Col. 12 Runoff	Q = (CA) (I) Q = (0.95) (7.1) Q = 6.75 ft^3/sec	Col. 8 times Col. 11
Col. 21 Slope	S = 0.001	Select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.001)^{0.5}]^{0.375}$ D = 1.96 ft D = 2.0 ft	Equation 3.2.18 or Figure 3.2-16 Use nominal size
Col. 14 Full Cap .	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46/(0.013)(2.0)^{2.67} (0.001)^{0.5}$ $Q_f = 7.12\ ft^3/s$	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_v/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67} (0.001)^{0.5}$ $V_f = 2.28$ ft/s	Equation 3.2.18 or Figure 3.2-16
Col. 16 Vel. Design	$Q/Q_f = 6.75/7.12 = 0.95$ $V/V_f = 1.15$ V = (1.15) (2.28) V = 2.6 ft/s	 Figure 3.2-18a
Col. 17 Sect. Time	$t_s = L/V = 14.1 / 2.6 / 60$ $t_s = 0.09$ min, use 0.0 min	Col. 3 divided by Col. 16
Col. 20 Crown Drop	$= H_{ah} = K_{ah} (V^2 / 2g)$ $= (1.5)(2.6)^2 / [(2)(32.2)]$ = 0.16 ft	Equation 3.2.36 and Table 3.2-13; $K_{ah} = 1.5$ for inlet - angled through 90 degrees
Col. 18 U/S Invert	= 344.23 - 0.16 = 344.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= 344.07 - (14.1)(0.001) = 344.06 ft	Col. 18 - (Col. 3)(Col. 21)

Structure 43 to 44

Col. 1 From	= 43	
Col. 2 To	= 44	
Col. 3 Run Length	L = 55.8 ft	Figure 3.2-30
Col. 4 Inlet Area	$A_i = 0.0$ ac	Figure 3.2-31
Col. 5 Total Area	$A_t = 1.31$ ac	Col. 4 plus structure 42 total area

Col. 6 "C"	C = n/a	Figure 3.2-31
Col. 7 Inlet CA	CA = 0.0	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.00 + 0.95 ΣCA = 0.95 ac	Col. 7 plus structure 42 total CA value
Col. 9 Inlet Time	n/a	No inlet
Col. 10 Sys. Time	t_c = 5 min	Col. 10 + Col. 17 for line 42-43
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21
Col. 12 Runoff	Q = (CA) I Q = (0.95) (7.1) Q = 6.75 ft^3/sec	Col. 8 times Col. 11
Col. 21 Slope	S = 0.01	Select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.01)^{0.5}]^{0.375}$ D = 1.27 ft D = 2.0 ft	Equation 3.2.18 or Figure 3.2-16 U/S conduit was 2.0 ft. - Do not reduce size inside the system
Col. 14 Full Cap.	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46)/(0.013)(2.0)^{2.67}(0.01)^{0.5}$ Q_f = 22.52 ft^3/s	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_v/n)\, D^{0.67} S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67}(0.01)^{0.5}$ V_f = 7.22 ft/s	Equation 3.2.17 or Figure 3.2-16
Col. 16 Vel. Design	Q/Q_f = 6.75/22.52 = 0.30 V/V_f = 0.84 V = (0.84) (7.22) V = 6.1 ft/s	Figure 3.2-18a
Col. 17 Sect. Time	t_s = 55.8 /6.1 / 60 t_s = 0.15 min, use 0.0 min	Col. 3 divided by Col. 16
Col. 19 D/S Invert	= 330.71 ft	Invert at discharge point in ditch
Col. 18 U/S Invert	= 330.71 + (55.8)(0.01) = 331.27 ft	Col. 19 + (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 344.06 - 331.27 = 12.79 ft straight run	Col. 19 previous run - Col. 18

(2) Energy Grade Line Evaluation Computations - English Units

The following computational procedure follows the steps outlined in Section 3.2.8.12 above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of

the computational process is provided in Figure 3.2-32 English and Figure 3.2-33 English. The column by column computations for each section of storm drain follow:

RUN FROM STRUCTURE 44 TO 43

Outlet

Step 1	Col. 1A	Outlet	
	Col. 14A	HGL = 333.0	Downstream pool elevation
	Col. 10A	EGL = 333.0	Assume no velocity in pool

Structure 44

Step 2	Col. 1A, 1B	Str. ID = 44	Outlet
	Col. 15A	Invert = 330.71 ft	Outfall invert
		TOC = 330.71 + 2.0	Top of storm drain at outfall
		TOC = 332.71	
		Surface Elev = 332.71	Match TOC
Step 3		HGL = TW = 333.0	From Step 1
		$EGL_i = HGL + V^2/2g$	Use Case 1 since TW is above the top of conduit
		$EGL_i = 333.0 + 0.07$	
	Col. 13A	$EGL_i = 333.07$	EGL_i for str. 44

Structure 43

Step 4	Col. 1A, 1B	Str. ID = 43	Next Structure
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 55.8 ft	Conduit length
Step 5	Col. 5A	$V = Q/A$	Velocity; use full barrel velocity since outlet is submerged.
		$V = 6.75/[(\pi/4)(2.0)^2]$	
		V = 2.15 ft/s	
	Col. 7A	$V^2/2g = (2.15)^2/(2)(32.2)$	Velocity head in conduit
		= 0.07 ft	
Step 6	Col. 8A	$S_f = [(Qn)/(K_Q D^{2.67})]^2$	Equation 3.2.18
		$S_f = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^2$	
		$S_f = 0.00090$ ft/ft	
Step 7	Col. 2B	$H_f = S_f L$	Equation 3.2.27
		$H_f = (0.0009)(55.8)$	Col. 8A x Col. 4A
		$H_f = 0.05$	
	Col. 7B &	$h_b, H_c, H_e, H_j = 0$	
	Col. 9A	Total = 0.05 ft	

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A
(English Solution)

COMPUTED BY _____ DATE _____ ROUTE _____
CHECKED BY _____ DATE _____ SECTION _____
PAGE _____ OF _____ COUNTY _____
INITIAL TAILWATER ELEV. _____

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K table B	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	(ft)	(ft^3/s)	(ft)	(fps)	(ft)		(ft)	(ft / ft)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16
OUTLET										333.00				333.00		
44													333.07		332.71	332.71
43	2.0	6.75	55.8	2.15	FULL	n/a	0.07	0.0009	0.05	333.12	0.5	0.04	333.16	*333.16	346.06	347.76
											*(Drop Structure)					
43	(New Outlet)			2.6		0.8	0.10						345.56	345.46	346.06	347.76
	(Above calculations are for inlet end of STR 43 for conduit 42-43)															
42	2.0	6.75	14.1	2.6	1.56	0.80	0.10	0.001	0.014	345.57	0.62	0.06	346.11	346.01	345.73	349.31
41	1.5	5.10	328.0	8.65	0.56	0.85	1.16	-	0	355.79	-	-	355.98	355.10	356.17	360.0
40	1.5	3.35	361.0	7.52	0.43	0.70	0.88		0	-	0	0	366.50	366.50	367.0	370.0

Figure 3.2-31 Energy Grade Line Computation Sheet, Table A, for English Example

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B
(English Solution)

COMPUTED BY _____ DATE _____ ROUTE _____
CHECKED BY _____ DATE _____ SECTION _____
PAGE _____ OF _____ COUNTY _____

	Pipe Losses (ft)						Structure Losses (ft)							
Str. ID	H_f	h_b	H_c	H_e	H_j	Total	d_{aho}	K_o	C_D	C_d	C_Q	C_p	C_B	K
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
44														
	0.05					0.05								
43							1.89							0.5
	0.014					0.014								
42							1.40	1.55	1.0	0.40	1.0	1.0	1.0	0.62
						0.0								
41														
														0.0
40							1.0							

Figure 3.2-32 Energy Grade Line Computation Sheet, Table B, for English Example

Step 8	Col. 10A	$EGL_o = EGL_i$ + pipe loss EGL_o = 333.07 + 0.05 EGL_o = 333.12 ft	
		HGL = 333.12 - 0.07 = 333.05	Check for full flow - close
		TOC = 331.27 + 2.0 = 333.27	Assumption OK
Step 9	Col. 8B	Not applicable due to drop structure	
Step 10	Col. 9B and 11A	K_e = 0.5	Inflow pipe invert much higher than d_{aho}. Assume square edge entrance
Step 17	Col. 12A	$K(V^2/2g)$ = (0.50)(0.07) $K(V^2/2g)$ = 0.04 ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o$ EGL_i = 333.12 + 0.04 EGL_i = 333.16 ft	Col 10A plus 12A
Step 19	Col. 14A	HGL = EGL_i = 333.16 ft	For drop structures, the HGL is the same as the EGL
		d_{aho} = HGL- invert = 333.16 - 331.27 = 1.89 ft	Col. 8B
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.06 + 2.0 U/S TOC = 346.06 ft	From storm drain comp. sheet (Figure 3.2-31)
Step 21	Col. 16A	Surf. Elev. = 347.76 ft 347.76 > 333.09	From Figure 3.2-30. Surface elev. exceeds HGL, OK
Step 2	Col. 1A, 1B Col. 15A Col. 16A	Str. ID = 43 U/S TOC = 344.06 + 2.0 = 346.06 Surface Elev = 347.76	Drop Structure - new start
Step 3		HGL' = inv. + $(d_c+D)/2$ HGL' = 344.06 + (0.80 + 2.0)/2	Calculate new HGL - Use Case 2 d_c from Figure 3.2-18b
	Col. 14A	HGL = 345.46 ft	
		EGL = HGL + $V^2/2g$	V = 2.6 fps from Prelim. Comp. Sht.
	Col. 13A	EGL = 345.46 + 0.10 EGL = 345.56 ft	

Structure 42

Step 4	Col.1A	Str. ID = 42	
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 14.1 ft	Conduit length
Step 5A	Col. 5A	V = 2.6 ft/s Q/Qf = 6.75 / 7.12 = 0.95	For flow: Actual velocity from storm drain computation sheet.
	Col. 6A	d_n = 1.56 ft Chart 26	Figure 3.2-31
	Col. 7A	$V^2/2g = (2.6)^2/(2)(32.2)$ $V^2/2g$ = 0.10 ft	Velocity head in conduit
Step 5B	Col.6bA	d_c = 0.80 ft	From HDS-5
Step 5C		$d_n < d_c$	Flow is subcritical
Step 6	Col. 8A	S_f = 0.001	Conduit not full so S_f = pipe slope d_n = 1.56 (Figure 3.2.18a) d_c = 0.80 (HDS-5) Flow is subcritical
Step 7		$H_f = S_f L$ H_f = (0.001) (14.1)	Equation 3.2.27 Col. 8A x Col. 5A
	Col. 2B	H_f = 0.014 ft h_b, H_c, H_e, H_j = 0	
	Col. 7B and 9A	Total = 0.014 ft	
Step 8		$EGL_o = EGL_i$ + total pipe loss EGL_o = 345.56 + 0.014	Col. 14A plus Col. 9A
	Col. 10A	EGL_o = 345.57 ft	
Step 9	Col. 8B	d_{aho} = EGL_o - velocity head - pipe invert d_{aho} = 345.57 - 0.10 - 344.07 d_{aho} = 1.40 ft	Col. 10A - Column 7A - pipe invert
Step 10	Col. 9B	$K_o = 0.1(b/D_o)(1-\sin\theta)+1.4(b/D_o)^{0.15}\sin(\theta)$ b = 4.0 ft D_o = 2.0 ft $\theta = 90^o$ $K_o = 0.1(4.0/2.0)(1 - \sin 90)+ 1.4(4.0/2.0)^{0.15}\sin 90$ K_o = 1.55	Equation 3.2.38 Access hole diameter. Col. 2A - outlet pipe diam Flow deflection angle
Step 11	Col. 10B	$C_D = (D_o/D_i)^3$ d_{aho} = 1.40 d_{aho}/D_o = (1.40/2.0) d_{aho}/D_o = 0.70 < 3.2 C_D = 1.0	Equation 3.2.39; pipe diameter Column 8B therefore

Step 12	Col. 11B	$C_d = 0.5\ (d_{aho}/\ D_o)^{0.6}$ $d_{aho}/D_o = 0.70 < 3.2$ $C_d = 0.5\ (1.4/2.0)^{0.6}$ $C_d = 0.40$	Equation 3.2.40; Flow depth correction.
Step 13	Col. 12B	$C_Q = (1\text{-}2 \sin \theta)(1\text{-}Q_i/Q_o)^{0.75}+1$ $C_Q = 1.0$	Equation 3.2.41; relative flow No additional pipes entering
Step 14	Col. 13B	$C_p = 1 + 0.2(h/D_o)[(h\text{-}d)/D_o]$ $C_p = 1.0$	Equation 3.2.42; plunging flow No plunging flow
Step 15	Col. 14B	$C_B = 1.0$	Benching Correction, flat floor (Table 3.2-15)
Step 16	Col. 15B and 11A	$K = K_o\ C_D\ C_d\ C_Q\ C_p\ C_B$ $K = (1.55)(1.0)(0.40)(1.0)(1.0)(1.0)$ $K = 0.62$	Equation 3.2.37
Step 17	Col. 12A	$K(V^2/2g) = (0.62)(0.10)$ $K(V^2/2g) = 0.06$ ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o + K(V_2/2g)$ $EGL_i = 346.05 + 0.06$ $EGL_i = 346.11$	Col. 10A plus 12A
Step 19	Col. 14A	$HGL = EGL_i - V^2/2g$ $HGL = 346.11 - 0.10$ $HGL = 346.01$ ft	Col. 13A minus Col. 7A
Step 20	Col 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.23 + 1.5 U/S TOC = 345.73 ft	Information from storm drain comp. sheet (Figure 3.2-31)
Step 21	Col 16A	Surf. Elev. = 349.31 ft 349.31 > 345.96	From Figure 3.2-30 Surface elev. exceeds HGL, OK

Structure 41

Step 4	Col. 1A, 1B Col. 2A Col. 3A Col. 4A	Str. ID = 41 D = 1.50 ft Q = 5.10 cfs L = 328 ft	Next Structure Pipe Diameter Conduit discharge (design value) Conduit length
Step 5	Part full flow from column's 12 and 15 of storm drain computation sheet.		Continue with Step 5A
Step 5A		$Q/Q_f = 5.1/18.1 = 0.28$ $d/d_f = 0.37$ $d = (0.37)\ (1.5)$	Figure 3.2-18a
	Col. 6aA	$d = 0.56$ ft	
		$V/V_f = 0.84$ $V = (0.84)(10.3)$	Figure 3.2-18a
	Col. 5A	$V = 8.65$ fps	

		$V^2/2g = (8.65)2/(2)(32.2)$	Velocity head
	Col. 7A	$V^2/2g = 1.16$ ft	
Step 5B	Col. 6bA	$d_c = 0.85$ ft	Figure 3.2-18b
Step 5C		0.56 < 0.85	Supercritical flow since $d_n < d_c$
Step 5D	Col. 7B	Total pipe loss = 0	

Structure 40

Step 5E	Col. 1A,1B	Str. Id. = 40	Next structure
	Col. 2A	D = 1.5 ft	Pipe diameter
	Col. 3A	Q = 3.35 cfs	Conduit discharge (design)
	Col. 4A	L = 361.0 ft	Conduit length
Step 5F		$Q/Q_f = 3.3/18.1 = 0.18$	
		$d/d_c = 0.29$	Figure 3.2-18a
		d = (0.29)(1.5)	
	Col. 6aA	d = 0.43 ft	
	Col. 6bA	$d_c = 0.7$ ft	Figure 3.2-18b
Step 5H		$V/V_f = 0.73$	Figure 3.2-18a
		V = (0.73)(10.3)	
	Col. 5A	V = 7.52 fps	
	Col. 7A	$V^2/2g = (7.52)^2/(2)(32.2)$	Velocity head
		$V^2/2g = 0.88$ ft	
Step 5I		$d_n = 0.43$ ft < 0.70 ft= d_c	Supercritical flow since $d_n < d_c$
Step 5K	Col. 11A, and 15B	K = 0.0	Str. 41 line; supercritical flow;
	Col. 12A	$K(V^2/2g) = 0$	no structure losses

Since both conduits 42-41 and 41-40 are supercritical - establish HGL and EGL at each side of access hole 41.

		HGL = Inv. + d	
		HGL = 354.07 + 0.56	D/S Invert + Flow depth
		HGL = 354.63 ft	
		EGL = 354.63 + 1.16	
		HGL + velocity head	
	Col. 10A	EGL = 355.79 ft	EGL_o of Str.41
		HGL = 354.67 + 0.43	U/S invert + Flow depth
	Col. 14A	HGL = 355.10 ft	Highest HGL
		EGL = 355.10 + 0.88	HGL + velocity head
	Col. 13A	EGL = 355.98 ft	EGL_i of Str. 41
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain comp
		U/S TOC = 354.67 + 1.5	Sheet (Figure 3.2-31) for Str. 41
		U/S TOC = 356.17 ft	
Step 21	Col. 16A	Surf. Elev. = 360.0 ft	From Figure 3.2-30.
		360.0 > 355.10	Surface elev. > HGL, OK

Step 10b	Col. 8B	d_{aho} = 0.67 (1.5) = 1.0 ft HGL = Str. 40 Inv. + d_{aho} HGL = 365.50 + 1.0.	Figure 3.3-2a, HW/D = 0.67 Structure Inv. from storm drain comp. sheet
	Col. 14A	HGL = 366.50 ft	
	Col.13A	EGL = 366.50 ft	Assume no velocity in str.
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 365.5 + 1.5 U/S TOC = 367.0 ft	Information from storm drain comp. sheet (Figure 3.2-31) for Str. 40
Step 21	Col. 16A	Surf. Elev. = 370.0 ft 370.0 ft > 366.50 ft	From Figure 3.2-30 Surface Elev. > HGL, OK

See Figures 3.2-32 and 3.2-33 for the tabulation of results. The final HGL values are indicated in Figure 3.2-30.

References

American Association of State Highway and Transportation Officials, 1981 and 1998. Model Drainage Manual.

American Association of State Highway and Transportation Officials, 1991. Model Drainage Manual. "Chapter 13: Storm Drainage Systems," AASHTO, Washington, D.C.

American Society of Civil Engineers, 1992. Design and Construction of Urban Stormwater Management Systems. "ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20." New York, N.Y.

Chang, F. M., Kilgore, R. T., Woo, D. C., and Mistichelli, M.P., Energy Losses Through Junction Access Holes, Volume I: Research Report and Design Guide, Federal Highway Administration, FHWA-RD-94-090, McLean, VA, 1994.

City of Austin, July 2003, Drainage Criteria Manual, Austin, Texas.

Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators. Hydraulic Engineering Circular No. 14, FHWA-EPD-86-11, U.S. Department of Transportation, Washington, D.C.

Federal Highway Administration, Revised 1993. Urban Drainage Design Participant Notebook, Course No. 13027. United States Department of Transportation, Federal Highway Administration, National Highway Institute Publication No. FHWA HI-89- 035, Washington, D.C.

Ickert, R. O. and Crosby, E. C., PhD, PE, 2003, “Storm Water Flow on a Curbed TxDOT Type Concrete Roadway,” Proceedings of the Texas Section, American Society of Civil Engineers, Spring 2003 Meeting, Corpus Christi, Texas, April 2 - 5.

Izzard, C. F., 1946, “Hydraulics of Runoff from Developed Surfaces,” Proceedings of the Twenty-Sixth Annual Meeting, Division of Engineering and Industrial Research, National Research Council, Highway Research Board, Vol. 26, pp. 129-150, Washington, D.C., December 5 - 8.

Normann, J. M., Houghtalen, R. J., and Johnston, W. J., 1985. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5, Federal Highway Administration, FHWA-IP-85-15, McLean, VA.

Texas Department of Transportation, November 2002, Hydraulic Design Manual, Austin, Texas, page 5-17.

U.S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12.

U.S. Department of Transportation, Federal Highway Administration, 1985, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, FHW-IP-85-15, McLean, VA.

U.S. Department of Transportation, Federal Highway Administration, 2001, “Urban Drainage Design Manual,” Hydraulic Engineering Circular No. 22, Second Edition, FHW-NHI-01-021, Washington, D.C.

Young, G. K. and Krolak, J. S., 1994. HYDRAIN-Integrated Drainage Design Computer System, Version 5.0. Federal Highway Administration, FHWA-RD-92- 061, McLean, Virginia.

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CHAPTER

4

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CHAPTER

4

HYDRAULIC DESIGN OF OPEN CHANNELS, CULVERTS, BRIDGES, AND DETENTION STRUCTURES

Section 4.1
Open Channels, Bridges, and Detention Structure Design Overview

4.1.1 Storm Water System Design

4.1.1.1 Introduction

Storm water system design is an integral component of both site and overall storm water management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on storm water runoff.

Storm water collection systems must be designed to provide adequate surface drainage while at the same time meeting other storm water management goals such as water quality, streambank protection, habitat protection, and groundwater recharge.

4.1.1.2 System Components

The storm water system components consist of all the *integrated* site design practices and storm water controls utilized on the site. Three considerations largely shape the design of the storm water systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove storm water from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, storm water ponds and wetlands, and small underground pipe systems which collect storm water runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger storm water systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The storm water (major) system consists of natural waterways, open channels, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The storm water system includes not only the trunk line system that receives the water, but also the natural overland relief which

functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This chapter is intended to provide design criteria and guidance on several on-site flood control system components, including culverts (Section 4.2), bridges (Section 4.3), vegetated and lined open channels (Section 4.4), storage design (Section 4.5), outlet structures (Section 4.6), and energy dissipation devices for outlet protection (Section 4.7). The rest of this section covers important considerations to keep in mind in the planning and design of storm water drainage facilities.

4.1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).

C. Check potential drainage outlets and methods, including:
   1. On-site (structural control, receiving water)
   2. Off-site (highway, storm drain, receiving water, regional control)
   3. Natural drainage system (swales)
   4. Existing drainage system (drain pipe)

D. Analyze areas for probable location of drainage structures and facilities.

E. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall storm water management system and plan.

4.1.2 Key Issues in Storm Water System Design

4.1.2.1 Introduction

The traditional design of storm water systems has been to collect and convey storm water runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Storm water systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural storm water controls to mitigate the major storm water impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in storm water system design.

4.1.2.2 General Design Considerations

- Storm water systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or storm water systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or storm water (minor) systems.
- It is important to ensure that the combined on-site flood control system and major storm water system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor storm water systems and/or major storm water structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of storm water systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major storm water system design capacity.

4.1.2.3 Culverts

- Culverts can serve double duty as flow retarding structures in grass channel design. Care should be taken to design them as storage control structures if depths exceed several feet, and to ensure safety during flows.
- Improved entrance designs can absorb considerable slope and energy for steeper sloped designs, thus helping to protect channels.

4.1.2.4 Bridges

- Bridges enable streams to maintain flow conveyance.
- Bridges are usually designed so that they are not submerged.
- Bridges may be vulnerable to failure from flood-related causes.
- Flow velocities through bridge openings should not cause scour within the bridge opening or in the stream reaches adjacent to the bridge.

4.1.2.5 Open Channels

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.
- Channels can be designed with natural meanders improving both aesthetics and pollution removal through increased contact time.
- Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress and super elevation.

- Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 4.1-1 illustrates a compound section that carries the 2-year and 100-year flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 1:12 slope to ensure drainage.

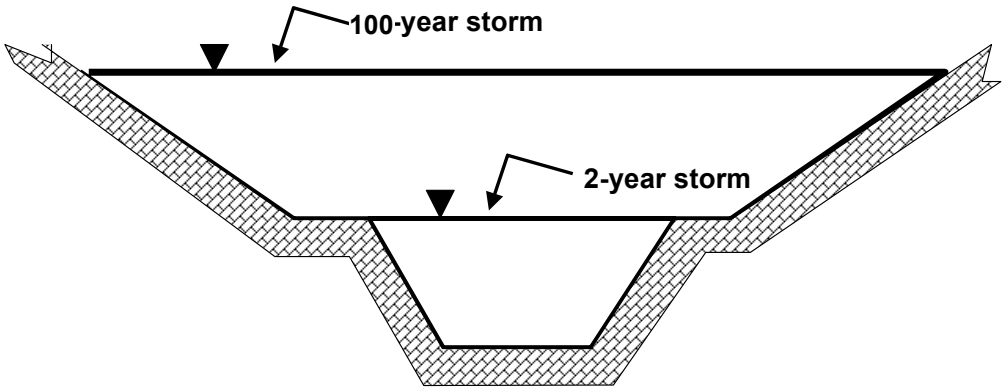

Figure 4.1-1 Compound Channel

- Flow control structures can be placed in the channels to increase residence time. Higher flows should be calculated using a channel slope from the top of the cross piece to the next one if it is significantly different from the channel bottom for normal depth calculations. Channel slope stability can also be ensured through the use of grade control structures that can serve as pollution reduction enhancements if they are set above the channel bottom. Regular maintenance is necessary to remove sediment and keep the channels from aggrading and losing capacity for larger flows.

4.1.2.6 Storage Design

- Storm water storage within a storm water system is essential to providing the extended detention of flows for water quality treatment and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection.
- Runoff storage can be provided within an on-site flood control system through the use of structural storm water controls and/or nonstructural features.
- Storm water storage can be provided by detention, extended detention, or retention.
- Storage facilities may be provided on-site, or as regional facilities designed to manage storm water runoff from multiple projects.

4.1.2.7 Outlet Structures

- Outlet structures provide the critical function of the regulation of flow for structural storm water controls.
- Outlet structures may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.
- Smaller, more protected outlet structures should be used for water quality and streambank protection flows.
- Large flows, such as flood flows, are typically handled through a broad crested weir, a riser with different sized openings, a drop inlet structure, or a spillway through an embankment.

4.1.2.8 Energy Dissipators

- Energy dissipators should be designed to return flows to non-eroding velocities to protect downstream channels.
- Care must be taken during construction that design criteria are followed exactly. The designs presented in this Manual have been carefully developed through model and full-scale tests. Each part of the criteria is important to their proper function.

4.1.3 Design Storm Recommendations

Listed below are the design storm recommendations for various storm water drainage system components to be designed and constructed in accordance with the minimum storm water management standards. Some jurisdictions may require that the design of storm water conveyance systems be sized for other storm frequencies. Please consult your local review authority to determine the local requirements. It is recommended that the full build-out conditions be used to calculate flows for the design storm frequencies below.

Storm Drainage Systems

Including storm drainage systems and pipes that do not convey runoff under public roadways, sometimes called lateral closed systems.

- 10- to 25-year design storm (for pipe and culvert design)
- 10- to 25-year design storm (for inlet design)
- 100-year design storm (for sumped inlets, unless overflow facilities are provided)

Roadway Culvert Design

Cross drainage facilities that transport runoff under roadways.

- 25- to 100-year design storm, or in accordance with TxDOT requirements, whichever is more stringent. (Criteria to be taken into consideration when selecting design flow include: roadway type, tailwater or depth of flow over road, structures, and property subject to flooding, emergency access, and road replacement costs.)

Bridge Design

Cross drainage facilities with a span of 20 feet or larger.

- 100-year design storm

Open Channel Design

Open channels include all channels, swales, etc.

- 100-year design storm

Channels may be designed with multiple stages (e.g., a low flow channel section containing the 2-year to 5-year flows, and a high flow section that contains the design discharge) to improve stability and better mimic natural channel dimensions. Where flow easements can be obtained and structures kept clear, overbank areas may also be designed as part of a conveyance system wherein floodplain areas are designed for storage and/or conveyance of larger storms.

Energy Dissipation Design

Includes all outlet protection facilities. All drainage system outlets, whether for closed conduits, culverts, bridges, open channels, or storage facilities, should provide energy dissipation to protect the receiving conveyance element from erosion.

- 100-year design storm (major systems)

Check Storm

Used to estimate the runoff that is routed through the drainage system and storm water management facilities to determine the effects on the facilities, adjacent property, floodplain encroachment, and downstream areas.

- 100-year design storm

References

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

Prince George's County, MD, 1999. Low-Impact Development Design Strategies, An Integrated Design Approach.

The Dewberry Companies, 2002, 2nd Edition, Land Development Handbook, McGraw-Hill Companies, Inc., New York, NY.

American Society of Civil Engineers, 1993, Design and Construction of Urban Stormwater Management Systems, Manual and Report No. 77.

Larry W. Mays, Editor, 2001, Stormwater Collection Systems Design Handbook, McGraw-Hill Companies, Inc., New York, NY.

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Section 4.2
Culvert Design

4.2.1 Overview

A *culvert* is a short, closed (covered) conduit that conveys storm water runoff under an embankment or away from the street right-of-way. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows.

The hydraulic and structural designs of a culvert must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

4.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 4.2-1 will be used. These symbols were selected because of their wide use.

Table 4.2-1 Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel depth	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
H_f	Depth of pool or head, above the face section of invert	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	-
L	Length of culvert	ft
N	Number of barrels	-
Q	Rate of discharge	cfs
S	Slope of culvert	ft/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

4.2.3 Design Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable.

4.2.3.1 Frequency Flood

See Section 4.1 or the local review authority for design storm requirements for the sizing of culverts.

The 100-year frequency storm shall be routed through all culverts to be sure building structures (e.g., houses, commercial buildings) are not flooded or increased damage does not occur to the highway or adjacent property for this design event.

4.2.3.2 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. The maximum allowable velocity for corrugated metal pipe is 15 feet per second. There is no specified maximum allowable velocity for reinforced concrete pipe, but outlet protection shall be provided where discharge velocities will cause erosion problems. To ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 feet per second, for the 1-year flow, when the culvert is flowing partially full is required

4.2.3.3 Buoyancy Protection

Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.

4.2.3.4 Length and Slope

The culvert length and slope should be chosen to approximate existing topography and, to the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment. The maximum slope using concrete pipe is 10% and for CMP is 14% before pipe-restraining methods must be taken. Maximum vertical distance from throat of intake to flowline in a drainage structure is 10 feet. Drops greater than 4 feet will require additional structural design.

4.2.3.5 Debris Control

In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled *Debris Control Structures* be consulted.

4.2.3.6 Headwater Limitations

Headwater is water above the culvert invert at the entrance end of the culvert. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway and is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The following criteria related to headwater should be considered:

- The *allowable headwater* is the depth of water that can be ponded at the upstream end of the culvert during the design flood, which will be limited by one or more of the following constraints or conditions:
  1. Headwater be non-damaging to upstream property
  2. Ponding depth be no greater than the low point in the road grade unless overflow has been allowed by the roadway design or at the applicable design criteria, such as the 5-year or 10-year flood level.

3. Ponding depth be no greater than the elevation where flow diverts around the culvert
4. Elevations established to delineate floodplain zoning
5. 18-inch (or applicable) freeboard requirements

- The headwater should be checked for the 100-year flood to ensure compliance with flood plain management criteria and for most facilities the culvert should be sized to maintain flood-free conditions on major thoroughfares with 18-inch freeboard at the low-point of the road.
- The maximum acceptable outlet velocity should be identified (see subsection 4.4.3).
- Either the headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided where these velocities are exceeded.
- In general, the constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.
- Other site-specific design considerations should be addressed as required.

4.2.3.7 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions. The following conditions must be considered:

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.
- For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined. See Section 4.4, *Open Channel Design*.
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body may establish the culvert tailwater.

4.2.3.8 Storage

If storage is being assumed or will occur upstream of the culvert, refer to subsection 4.2.4.6 regarding storage routing as part of the culvert design.

4.2.3.9 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 4.2-2.

4.2.3.10 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all metal culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall.

This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

4.2.3.11 Wingwalls and Aprons

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

4.2.3.12 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.

4.2.3.13 Material Selection

Reinforced concrete pipe (RCP), pre-cast and cast in place concrete boxes are recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP and fully coated corrugated metal pipe can be used in all other cases. High-density polyethylene (HDPE) pipe may also be used as specified in the municipal regulations. Table 4.2-3 gives recommended Manning's n values for different materials.

4.2.3.14 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

4.2.3.15 Culvert Sizes

The minimum allowable pipe diameter shall be 18 inches.

4.2.3.16 Weep Holes

Weep holes are sometimes used to relieve uplift pressure on headwalls and concrete rip-rap. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels through the fill embankment. The filter materials should be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

Table 4.2-2 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting form fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Slide- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or [1/12(B)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

[1] Although laboratory tests have not been completed on K_e values for High-Density Polyethylene (HDPE) pipes, the K_e values for corrugated metal pipes are recommended for HDPE pipes.

* Note: "End Section conforming to fill slope", made of either metal or concrete, 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.

Source: HDS No. 5, 2001

Table 4.2-3 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corrugations	2 2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 2001

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

4.2.3.17 Outlet Protection

See Section 4.5 for information on the design of outlet protection.

4.2.3.18 Erosion and Sediment Control

Erosion and sediment control shall be in accordance with the latest approved Soil Erosion and Sediment Control Ordinance for the municipality.

4.2.3.19 Environmental Considerations

Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.

4.2.3.20 Safety Considerations

Roadside safety should be considered for culverts crossing under roadways. Guardrails or safety end treatments may be needed to enhance safety at culvert crossings. The AASHTO roadside design guide should be consulted for culvert designs under and adjacent to roadways.

4.2.4 Design Procedures

4.2.4.1 Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth:

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

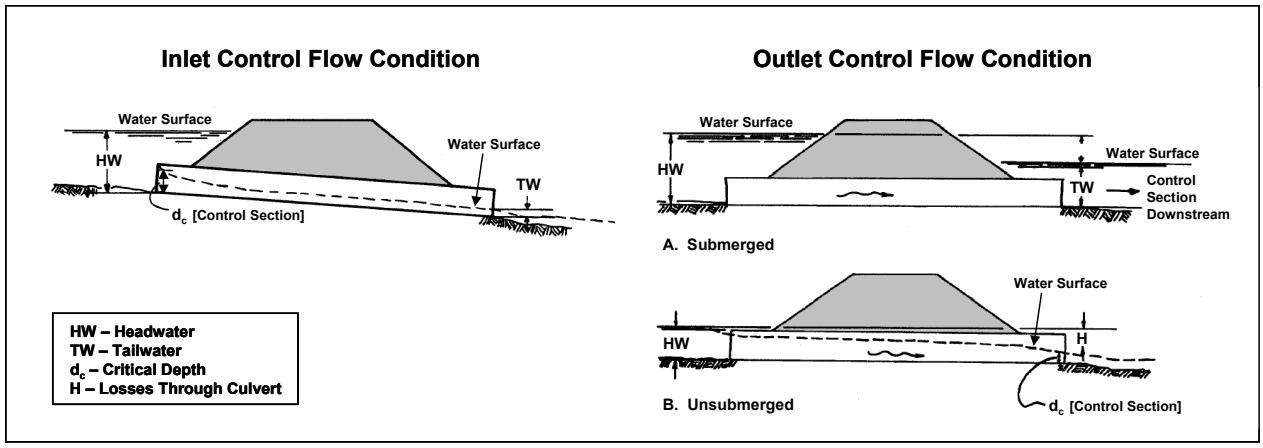

Figure 4.2-1 Culvert Flow Conditions
(Adapted from: HDS-5, 2001)

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001.

4.2.4.2 Procedures

The culvert design process includes the following basic stages:

1. Define the location, orientation, shape, and material for the culvert to be designed. In many instances, consider more than single shape and material.
2. With consideration of the site data, establish allowable outlet velocity and maximum allowable depth of barrel.
3. Based on upon subject discharges, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (culvert hydraulic length, entrance conditions, and conduit shape and material).
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.
6. Treat any excessive outlet velocity separately from headwater.

There are three procedures for designing culverts: inlet control design equations, manual use of inlet and outlet control nomographs, and the use computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

4.2.4.3 Inlet Control Design Equations

This section contains explanations of the equations and methods used to develop the design charts in HDS No. 5, where those equations and methods are not fully described in the main text. The following topics are discussed: the design equations for the unsubmerged and submerged inlet control nomographs, the dimensionless design curves for culvert shapes and sizes without nomographs, and the dimensionless critical depth charts for long span culverts and corrugated metal box culverts.

Inlet Control Nomograph Equations: The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. Of these, the first and fourth through seventh reports dealt with the hydraulics of pipe and box culvert entrances, with and without tapered inlets (4, 7, to 10). These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs (56 and 57).

The two basic conditions on inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged, the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Table 4.2-4 contains the unsubmerged and submerged inlet control design equations. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with tow correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs.

The constants and the corresponding equation form are given in Table 4.2-5. Table 4.2-5 is arranged in the same order as the design nomographs in section 4.2.4.4, and provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

The equations may be used to develop design curves for any conduit shape or size. Careful examination of the equation constants for a given form of equation reveals that there is very little difference between the constants for a given inlet configuration. Therefore, given the necessary conduit geometry for a new shape from the manufacturer, a similar shape is chosen from Table 4.2-5, and the constants are used to develop new design curves. The curves may be quasi-dimensionless, in terms of $Q/AD^{0.5}$ and HW_i/D, or dimensional, in terms of Q and HW_i for a particular conduit size. To make the curves truly dimensionless, $Q/AD^{0.5}$ must be divided by $g^{0.5}$, but this results in small decimal numbers. Note that coefficients for rectangular (Box) shapes should not be used for nonrectangular (circular, arch, pipe-arch, etc.) shapes and vice-versa. A constant slope value of 2 percent (0.02) is usually selected for the development of

design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent (except for mitered inlets).

Table 4.2-4 Inlet Control Design Equations

Unsubmerged*		
Form (1)	$\frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{K_u Q}{AD^{0.5}}\right)^M - 0.5S$ ***	**(4.2.1)**
Form (2)	$\frac{HW_i}{D} = K\left(\frac{K_u Q}{AD^{0.5}}\right)^M$	**(4.2.2)**
Submerged**		
	$\frac{HW_i}{D} = c\left(\frac{K_u Q}{AD^{0.5}}\right)^2 + Y - 0.5S$ ***	**(4.2.3)**
Definitions		
HW_i	Headwater depth above inlet control section invert, m (ft)	
D	Interior height of culvert barrel, m (ft)	
H_c	Specific head at critical depth ($d_c + V_c^2/2g$), m^2 (ft^2)	
Q	Discharge, m^3/s (ft^3/s)	
A	Full cross sectional area of culvert barrel, m^2 (ft^2)	
S	Culvert barrel slope, m/m (ft/ft)	
K, M, c, Y	Constants from Table 4.2-5	
K_u	1.811 SI (1.0 English)	
* Equations 4.2.1 and 4.2.2 (unsubmerged) apply to about $Q/AD^{0.5}$ = 1.93 (3.5 English)		
** Equation 4.2.3 (submerged) above applies to about $Q/AD^{0.5}$ = 2.21 (4.0 English)		
*** For mitered inlets use +0.7 S instead of -0.5 S as the slope correction factor.		

Table 4.2-5 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
1	Circular Concrete	1	Square edge w/ headwall	1	.0098	2.0	.0398	.67	56/57
		2	Groove end w/ headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69	56/57
		2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7° bevels		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30° to 75° wingwall flares	1	.026	1.0	.0347	.81	56
		2	90° and 15° wingwall flares		.061	.75	.0400	.80	56
		3	0° wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45° wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18° to 33.7° wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90° headwall w/ 3/4" chamfers	2	.515	.667	.0375	.79	8
		2	90° headwall w/ 45° bevels		.495	.667	.0314	.82	
		3	90° headwall w/ 33.7° bevels		.486	.667	.0252	.865	

Table 4.2-5 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References*
11	Rectangular Box	1	3/4" chamfers; 45° skewed headwall	2	.545	.667	.04505	.73	8
		2	3/4" chamfers; 30° skewed headwall		.533	.667	.0425	.705	
		3	3/4" chamfers; 15° skewed headwall		.522	.667	.0402	.68	
		4	45° bevels; 10°-45° skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4" chamfers	1	45° non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4° non-offset wingwall flares		.493	.667	.0361	.806	
		3	18.4° non-offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71	
13	Rectangular Box Top Bevels	1	45° wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7° wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4° wingwall flares - offset		.493	.667	.0227	.887	
16-19	CM Boxes	2	90° headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	
29	Horizontal Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
30	Vertical Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch 18" Corner Radius CM	1	90° headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch 18" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
		2	No Bevels		.0088	2.0	.0368	.68	
		3	33.7° Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch 31" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
			No Bevels		.0088	2.0	.0368	.68	
			33.7° Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Thin wall projecting		.0340	1.5	.0496	.57	
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.90	3
		2	Rough tapered inlet throat		.519	.64	.0210	.90	
56	Elliptical Inlet Face	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
		2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75	
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular Concrete	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
		2	Side tapered-more favorable edges		.56	.667	.0378	.87	
59	Rectangular Concrete	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
			Slope tapered-more favorable edges		.50	.667	.0378	.71	

* These references are cited in FHWA, 2001, HYD-5. They can be accessed at the Federal Highway Administration web site: www.fhwa.dot.gov/bridge/hydpub.htm.

4.2.4.4 Nomographs

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figures 4.2-2(a) and (b) show examples of an inlet control and outlet control nomographs for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001, Second Edition.

This section presents design guidance for culverts originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

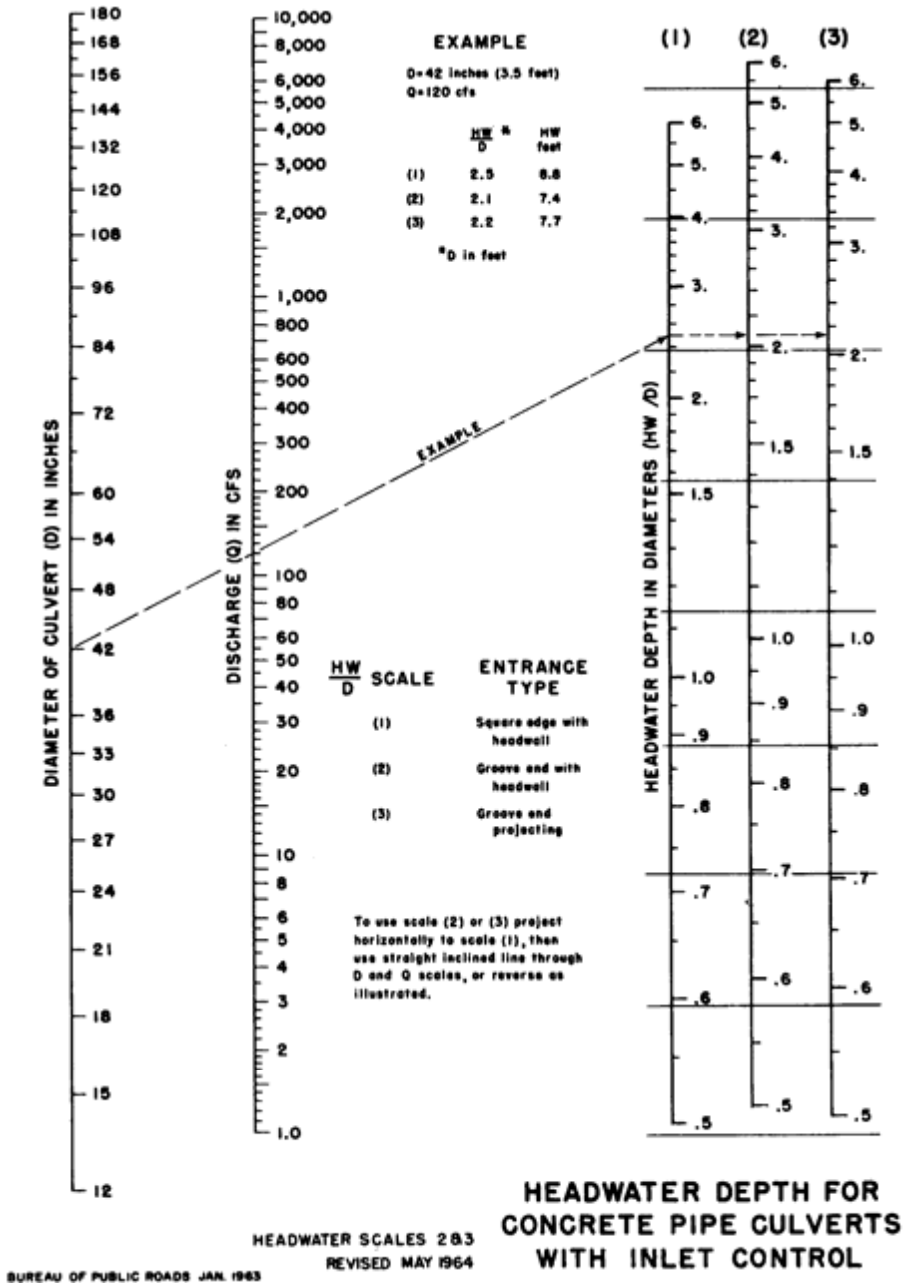

Figure 4.2-2(a) Headwater Depth for Concrete Pipe Culvert with Inlet Control

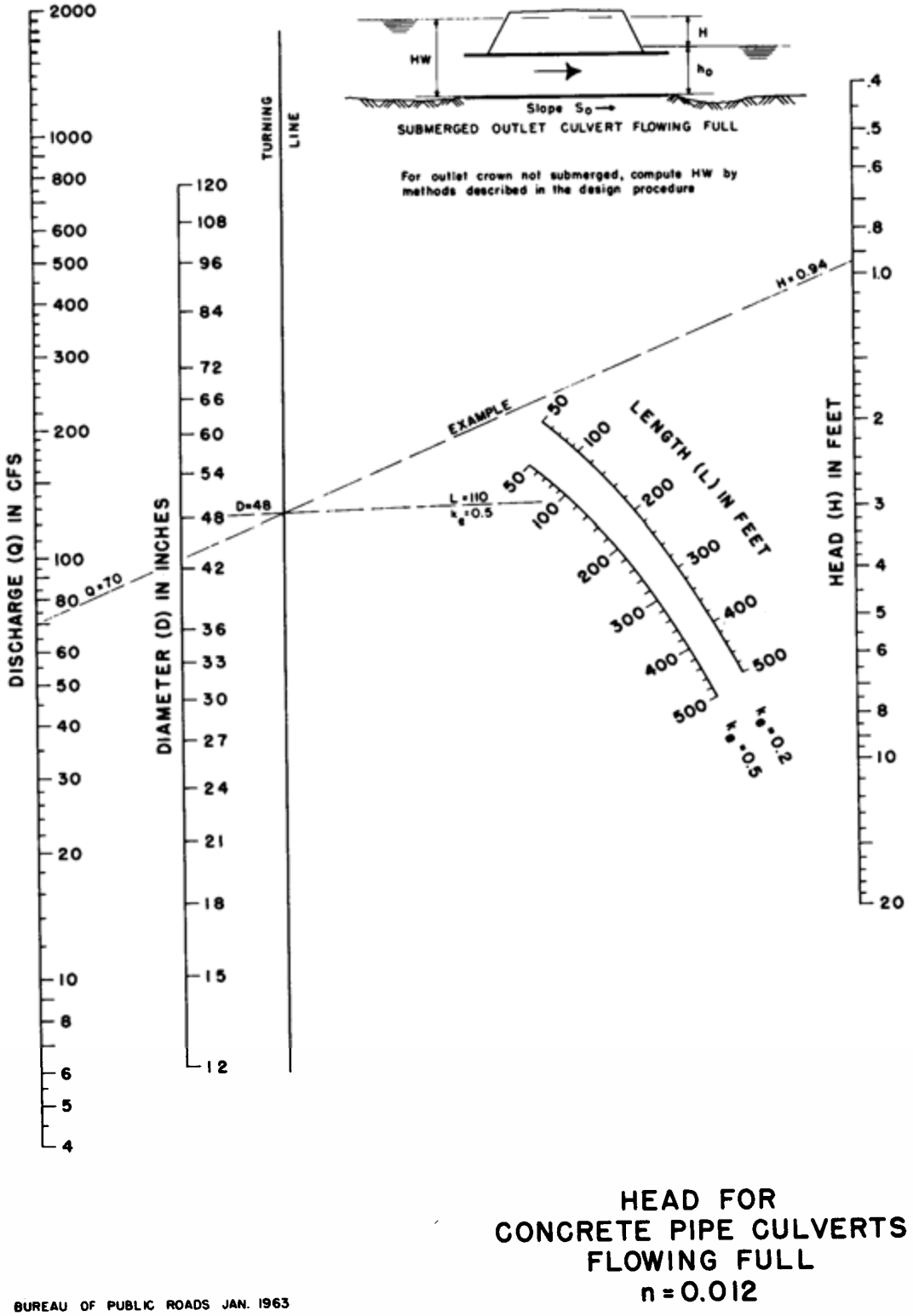

Figure 4.2-2(b) Head for Concrete Pipe Culverts Flowing Full

4.2.4.5 Design Procedure

The following design procedure requires the use of inlet and outlet nomographs.

Step 1 List design data:

Q = discharge (cfs)
L = culvert length (ft)
S = culvert slope (ft/ft)
TW = tailwater depth (ft)
V = velocity for trial diameter (ft/s)
K_e = inlet loss coefficient
HW = allowable headwater depth for the design storm (ft)

Step 2 Determine trial culvert size by assuming a trial velocity of 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

Step 3 Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

$$\mathbf{HW = H + h_o - LS} \qquad \mathbf{(4.2.4)}$$

where:

h_o = ½ (critical depth + D), or tailwater depth, whichever is greater
L = culvert length
S = culvert slope

Step 4 Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

Step 5 Calculate exit velocity and if erosion problems might be expected, refer to Section 4.7 for appropriate energy dissipation designs. Energy dissipation designs may affect the outlet hydraulics of the culvert.

4.2.4.6 Performance Curves - Roadway Overtopping

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the use of computer programs.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step 1 Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

Step 2 Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 4.2.5 to calculate flow rates across the roadway.

$$Q = C_d L (HW)^{1.5} \qquad (4.2.5)$$

where:

- Q = overtopping flow rate (ft^3/s)
- C_d = overtopping discharge coefficient
- L = length of roadway (ft)
- HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 4.2-3 on the next page for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 44 in HDS No. 5, 2001.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

4.2.4.7 Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See subsection 4.2.7 and Section 2.2 for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, 2001, Federal Highway Administration, pages 123 - 142.

Note: Storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.

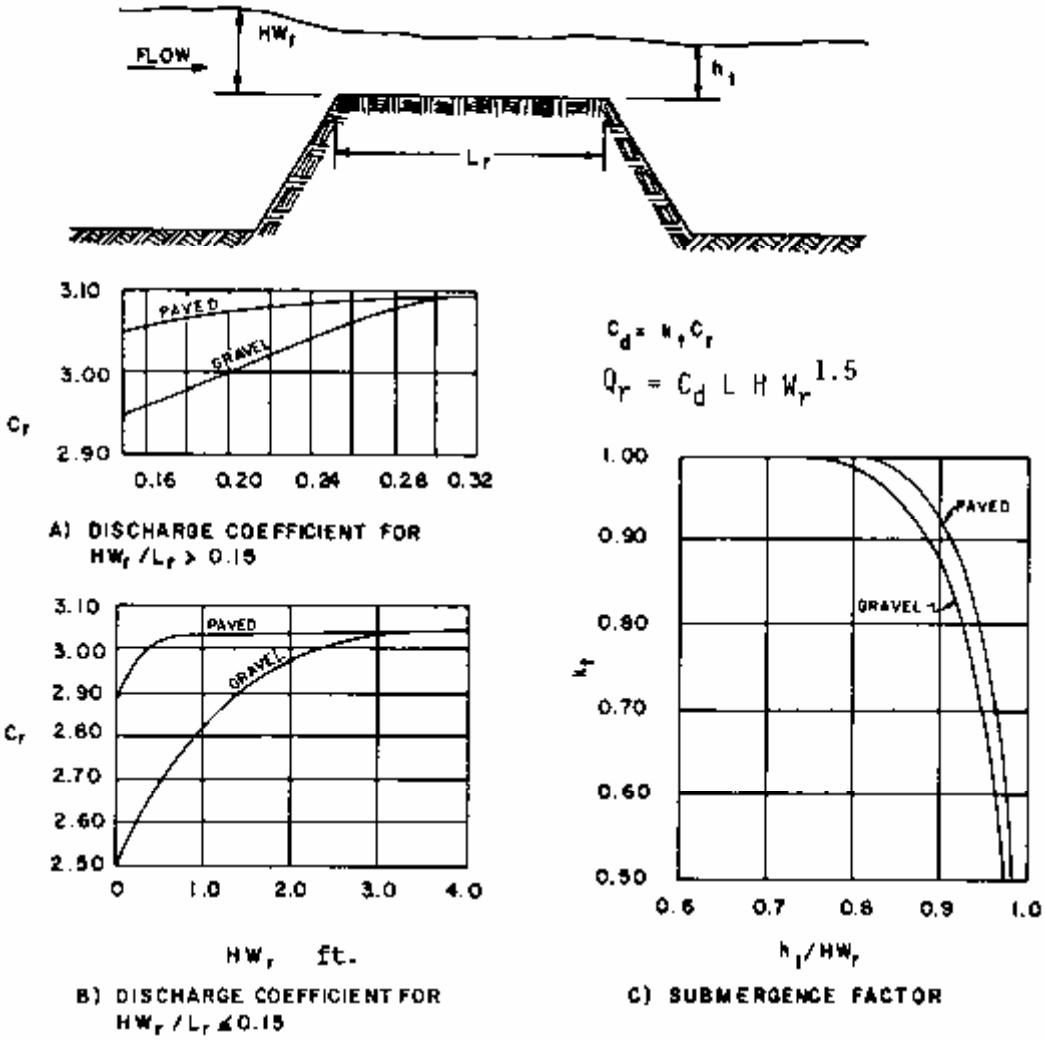

Figure 4.2-3 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 2001)

4.2.5 Culvert Design Example

4.2.5.1 Introduction

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

4.2.5.2 Example

Size a culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

4.2.5.3 Example Data

Input Data

Discharge for 2-yr flood = 35 cfs

Discharge for 25-yr flood = 70 cfs

Allowable H_w for 25-yr discharge = 5.25 ft

Length of culvert = 100 ft

Natural channel invert elevations - inlet = 15.50 ft, outlet = 14.30 ft

Culvert slope = 0.012 ft/ft

Tailwater depth for 25-yr discharge = 3.5 ft

Tailwater depth is the normal depth in downstream channel

Entrance type = Groove end with headwall

4.2.5.4 Computations

1. Assume a culvert velocity of 5 ft/s. Required flow area = 70 cfs/5 ft/s = 14 ft^2 (for the 25-yr recurrence flood).
2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = $(3.14D^2)/4$ or D = (Area times $4/3.14)^{0.5}$. Therefore: D = $((14 \text{ sq ft} \times 4)/3.14)^{0.5} \times 12$ in/ft) = 50.7 in
3. A grooved end concrete culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 4.2-2(a)), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.
4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft, which is less than the allowable headwater of 5.25 ft. Since 3.72 ft is considerably less than 5.25 try a small culvert.
5. Using the same procedures outlined in steps 4 and 5 the following results were obtained.

   42-inch culvert – HW = 4.13 ft

   36-inch culvert – HW = 5.04 ft

   Select a 36-inch culvert to check for outlet control.
6. The culvert is checked for outlet control by using Figure 4.2-2(b).

   With an entrance loss coefficient K_e of 0.20, a culvert length of 100 ft, and a pipe diameter of 36 in., an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: $HW = H + h_o - LS$

   Compute h_o

   $h_o = T_w$ or ½ (critical depth in culvert + D), whichever is greater.

   h_o = 3.5 ft or h_o = ½ (2.7 + 3.0) = 2.85 ft

   Note: critical depth is obtained from Figure 3.2-18(b).

   Therefore: h_o = 3.5 ft

   The headwater depth for outlet control is:

   $HW = H + h_o - LS = 2.8 + 3.5 - (100) \times (0.012) = 5.10$ ft
7. Since HW for outlet control (5.10 ft) is greater than the HW for inlet control (5.04 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.

8. Estimate outlet exit velocity. Since this culvert is an outlet control and discharges into an open channel downstream with tailwater above culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the exit velocity will be:

   $Q = VA$

   Therefore: $V = 70 / (3.14(3.0)^2)/4 = 9.9$ ft/s

   With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See Section 4.7 (*Energy Dissipation Design)*.

9. Check for minimum velocity using the 2-year flow of 35 cfs.

   Therefore: $V = 35 / (3.14(3.0)^2/4 = 5.0$ ft/s > minimum of 2.5 - OK

10. The 100-year flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

Figure 4.2-4 provides a convenient form to organize culvert design calculations.

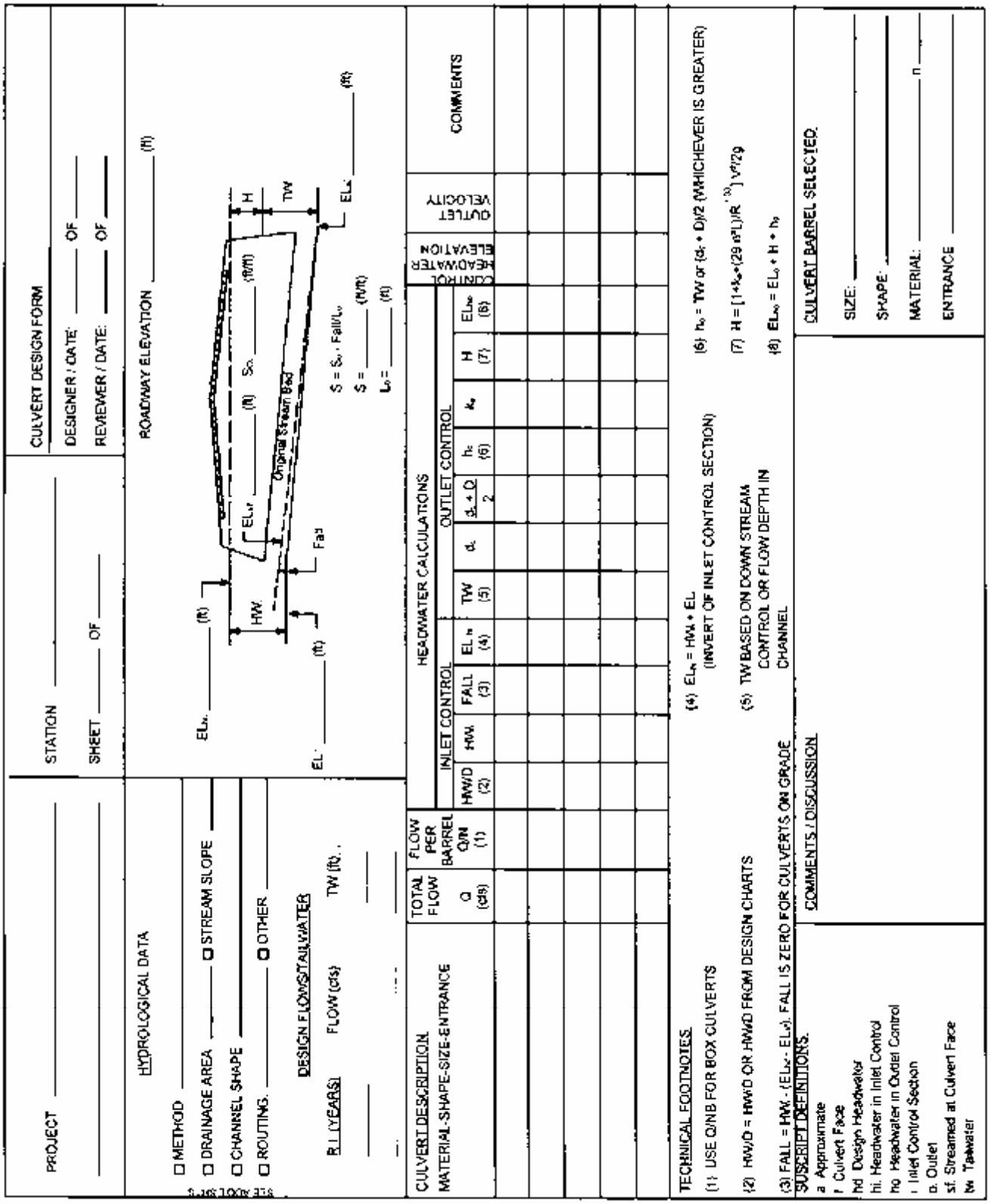

Figure 4.2-4 Culvert Design Calculation Form
(Source: HDS No. 5, 2001)

4.2.6 Design Procedures for Beveled-Edged Inlets

4.2.6.1 Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Design Series No. 5 entitled, Hydraulic Design of Highway Culverts.

4.2.6.2 Design Figures

Four inlet control figures for culverts with beveled edges are found in Appendix D of HDS No. 5.

Chart	Page	Use for
3	D-3A & B	circular pipe culverts with beveled rings
10	D-10A & B	90° headwalls (same for 90° wingwalls)
11	D-11A & B	skewed headwalls
112	D-12A & B	wingwalls with flare angles of 18 to 45 degrees

The following symbols are used in these figures:

B – Width of culvert barrel or diameter of pipe culvert

D – Height of box culvert or diameter of pipe culvert

H_f – Depth of pool or head, above the face section of invert

N – Number of barrels

Q – Design discharge

4.2.6.3 Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 in subsection 4.2.8 apply only to bevels having either a 33° angle (1.5:1) or a 45° angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 0.5 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft. For example, the minimum bevel dimensions for an 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches

Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches

For a 1.5:1 bevel computations would result in d = 6 and b = 8 inches.

4.2.6.4 Design Figure Limits

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft by 8 ft box culvert:

Top Bevel is proportioned based on the height of 8 feet, which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

Side Bevel is proportioned based on the clear width of 16 feet, which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

4.2.6.5 Multibarrel Installations

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

4.2.6.6 Skewed Inlets

It is recommended that Chart 11 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Skewed inlets (at an angle with the centerline of the stream) should be avoided whenever possible and should not be used with side- or slope-tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

4.2.7 Flood Routing and Culvert Design

4.2.7.1 Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is accepted without flood routing, then costly over-design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- The total area of flooding,
- The average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

4.2.7.2 Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that a computer program be used to perform routing calculations; however, an engineer should be familiar with the culvert flood routing design process.

A multiple trial and error procedure is required for culvert flood routing. In general:

Step 1 A trial culvert(s) is selected

Step 2 A trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected

Step 3 Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

Step 4 The hydraulic findings are compared to the selected site criteria

Step 5 If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

4.2.7.3 Comprehensive Design Guidance

Comprehensive design discussions and guidance may be found in the Federal Highway Administration, National Design Series No. 5, document entitled Hydraulic Design of Highway Culverts, Second Edition, published in 2001. This document is available from the National Technical Information Service as Item Number PB2003102411*DL. (http://www.ntis.gov/search.htm) Search for this document using the Item Number.

References

American Association of State Highway and Transportation Officials, 1982. Highway Drainage Guidelines.

American Association of State Highway and Transportation Officials, 1998. Model Drainage Manual.

Debo, Thomas N. and Andrew J. Reese, 1995. Municipal Storm Water Management. Lewis Publishers.

Federal Highway Administration, 1978. Hydraulics of Bridge Waterways. Hydraulic Design Series No. 1.

Federal Highway Administration, 2001. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5. (Downloadable from the Internet http://www.ntis.gov/search.htm, Item Number PB2003102411*DL)

Federal Highway Administration, 1971. Debris-Control Structures. Hydraulic Engineering Circular No. 9.

Federal Highway Administration, 1987. HY8 Culvert Analysis Microcomputer Program Applications Guide. Hydraulic Microcomputer Program HY8.

Federal Highway Administration, 2001. Urban Drainage Design Manual, Second Edition. Hydraulic Engineering Circular No. 22.

Federal Highway Administration Internet Site for Publications, http://www.fhwa.dot.gov/bridge/hypub.htm#hds.

HYDRAIN Culvert Computer Program (HY8). Available from McTrans Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.

HYDRAIN for Windows available at GKY and Associates, Inc., Springfield, Virginia (http://www.gky.com/HFW/.)

U. S. Department of Interior, 1983. Design of Small Canal Structures.

Section 4.3
Bridge Design

4.3.1 Overview

Bridges enable streams to maintain flow conveyance and to sustain aquatic life. They are important and expensive highway hydraulic structures vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing must be recognized and considered during the development, construction, and maintenance phases.

This section addresses structures designed hydraulically as bridges, regardless of length. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the floodplains.

The following subsections present considerations related to the hydraulics of bridges. It is generally excerpted from Chapter 9 of the Texas Department of Transportation (TxDOT) Hydraulics Design Manual dated March 2004.

4.3.1.1 Bridge Hydraulics Considerations

When beginning analysis for a cross-drainage facility, the flood frequency and stage-discharge curves should first be established, as well as the type of cross-drain facility. The choice is usually between a bridge and a culvert. Bridges are usually chosen if the discharge is significant or if the stream to be crossed is large in extent. Both types of facilities should be evaluated and a choice made based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

4.3.1.2 Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves:

- Determining the backwater associated with each alternative profile and waterway opening(s)
- Determining the effects on flow distribution and velocities
- Estimating scour potential

The hydraulic design of a bridge over a waterway involves the following such that the risks associated with backwater and increased velocities are not excessive:

- Establishing a location
- Bridge length
- Orientation
- Roadway and bridge profiles

A hydrologic and hydraulic analysis is recommended for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely affect the floodplain, even if no structural modifications are necessary. Typically, this should include the following:

- An estimate of peak discharge (sometimes complete runoff hydrographs)
- Existing and proposed condition water surface profiles for design and check flood conditions
- Consideration of the potential for stream stability problems and scour potential.

4.3.1.3 Freeboard

Navigational clearance and other reasons notwithstanding, the low chord elevation is defined as the sum of the design normal water surface elevation (high water) and a *freeboard*. For on system TxDOT bridges, TxDOT recommends a minimum freeboard of 2 ft to allow for passage of floating debris and to provide a safety factor for design flood flow. Higher freeboards may be appropriate over streams that are prone to heavy debris loads, such as large tree limbs, and to accommodate other clearance needs. Other constraints may make lower freeboards desirable, but the low chord should not impinge on the design high water. Generally, for off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced, unless the results of a risk assessment indicate a different structure is the most beneficial option.

4.3.1.4 Roadway/Bridge Profile

A bridge is integrated into both the stream and the roadway and must be fully compatible with both. Therefore, the alignment of the roadway and the bridge are the same between the ends of the bridge. Hydraulically, the complete bridge profile can be any part of the structure that stream flow can strike or impact in its movement downstream. If the stream gets high enough to inundate the structure, then all parts of the roadway and the bridge become part of the complete bridge profile.

For TxDOT design, the roadway must not be inundated by the design flood, but inundation by the 100-year flood is allowed. Unless the route is an emergency escape route, it is often desirable to allow floods in excess of the design flood to overtop the road. This helps minimize both the backwater and the required length of structure.

Several vertical alignment alternatives are available for consideration, depending on site topography, traffic requirements, and flood damage potential. The alternatives range from crossings that are designed to overtop frequently to crossings that are designed to rarely or never overtop.

4.3.1.5 Crossing Profile

The horizontal alignment of a highway at a stream crossing should be taken into consideration when selecting the design and location of the waterway opening as well as the crossing profile. Every effort should be made to align the highway so that the crossing will be normal to the stream flow direction (highway centerline perpendicular to the streamline).

Often, this is not possible because of the highway or stream configuration. When a skewed structure is necessary, it should be ensured that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow.

Bent caps should be oriented as near to the skew of the streamlines at flood stage as possible. Headers should be skewed to minimize eddy-causing obstructions. A relief opening may be provided to reduce the likelihood of trapped flow and minimize the amount of flow that would have to travel up against the general direction of flow along the embankment.

4.3.1.6 Single Versus Multiple Openings

For a single structure, the flow will find its way to an opening until the roadway is overtopped. If two or more structures have flow area available, after accumulating a head, the flow will divide and proceed to the structures offering the least resistance. The point of division is called a stagnation point.

In usual practice, the TxDOT recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft. If flow distances along the embankment are greater than recommended, an additional relief structure or opening should be considered. A possible alternative to the provision of an additional structure is a guide bank (spur dike) to control the turbulence at the header. Also, natural vegetation between the toe of slope and the right-of-way line is useful in controlling flow along the embankment. Therefore, special efforts should be made to preserve any natural vegetation in such a situation.

4.3.1.7 Factors Affecting Bridge Length

The discussions of bridge design assume normal cross sections and lengths (perpendicular to flow at flood stage). Usually one-dimensional flow is assumed, and cross sections and lengths are considered 90° to the direction of stream flow at flood stage.

If the crossing is skewed to the stream flow at flood stage, all cross sections and lengths should be normalized before proceeding with the bridge length design. If the skew is severe and the floodplain is wide, the analysis may need to be adjusted to offset the effects of elevation changes within the same cross section.

The following examples illustrate various factors that can cause a bridge opening to be larger than that required by hydraulic design.

- Bank protection may be placed in a certain location due to local soil instability or a high bank.
- Bridge costs may be cheaper than embankment costs.
- A highway profile grade line might dictate an excessive freeboard allowance. For sloping abutments, a higher freeboard will result in a longer bridge.
- High potential for meander to migrate, or other channel instabilities may warrant a longer opening.

4.3.2 Symbols and Definitions

The hydraulics of bridge openings are basically the same as those of open channel flow. Therefore, the symbols and definitions are essentially the same as those of Section 4.4.2 presented in Table 4.4-1. There are other definitions unique to bridges which are presented here. They are defined in the TxDOT Hydraulic Design Manual.

4.3.2.1 Flow Zones and Energy Losses

Figure 4.3-1 shows a plan of typical cross section locations that establish three flow zones that should be considered when estimating the effects of bridge openings.

Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. No detailed guidance is available, but a distance equal to about four times the length of the average embankment constriction is reasonable for most situations. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 4.3-2.

Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally, the bridge opening is obtained by superimposing the bridge geometry on cross sections 2 and 3.

Zone 3 represents an area from the upstream face of the bridge to a distance upstream where the contraction of flow must occur. A distance upstream of the bridge equal to the length of the average embankment constriction is a reasonable approximation of the location at which contraction begins. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the "approach" cross section.

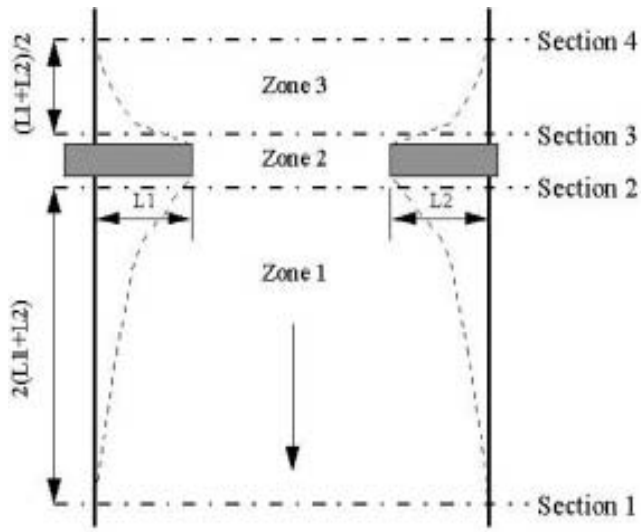

Figure 4.3-1 Flow Zones at Bridges
(TxDOT Hydraulic Design Manual)

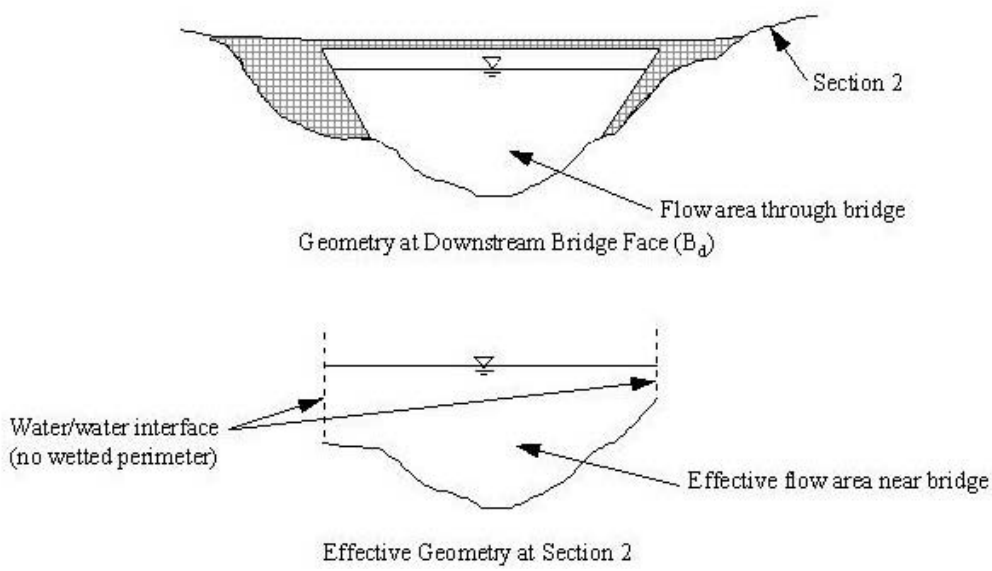

Figure 4.3-2 Effective Geometry for Bridge (Section 2 shown, Section 3 similar)
(TxDOT Hydraulic Design Manual)

4.3.2.2 Bridge Flow Class

The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow.

Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. (This condition should exist for the design frequency of all new on-system bridges.) Low flow is divided into categories as described in the "Low Flow Classes" table below Type I is the most common in Texas, although severe constrictions compared to the flow conditions could result in Types IIA and IIB. Type III is likely to be limited to steep hills and mountainous regions.

Low Flow Class	Description
I	Subcritical flow through all Zones
IIA	Subcritical flow through Zones 1 and 3; flow through critical depth in Zone 2
IIB	Subcritical flow through Zone 3; flow through critical depth in Zone 2, hydraulic jump in Zone 1
III	Supercritical flow through all Zones

High flow refers to conditions in which the water surface impinges on the bridge superstructure:

- When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate.
- When the tailwater submerges the low chord but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the tailwater overtops the roadway, neither sluice gate flow nor orifice flow is reasonable, and the flow is either weir flow or open flow.

4.3.3 Design Criteria

The design of a bridge should take into account many different engineering and technical aspects at the bridge site and adjacent areas. The following design criteria should be considered for all bridge designs as applicable. See the Local Criteria section of this manual for additional considerations.

4.3.3.1 Frequency Flood

Design discharges chosen by TxDOT for bridges vary with the functional classification and structure type. For major river crossings, a return period of 50 years is recommended. Flow small bridges, the recommended return period is 25 years. In all cases the check flood is for the 100-year return period.

4.3.3.2 Freeboard

Typical freeboard, the length between the computed design water surface and the low chord, is two feet. In urban settings, it may be prudent to use the 100-year fully-developed discharge to check the bridge design. The 100-year flood discharge, assuming blockage of outlet works, with 6" of freeboard. Some municipalities may specify different design storms and freeboard requirements.

4.3.3.3 Loss Coefficients

The contraction and expansion of water through the bridge opening creates hydraulic losses. These losses are accounted for through the use of loss coefficients. Table 4.3-1 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 4.3-1 Recommended Loss Coefficients for Bridges		
Transition Type	**Contraction (K_c)**	**Expansion (K_e)**
No losses computed	0.0	0.0
Gradual transition	0.1	0.3
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

4.3.4 Design Procedures

The following is a general bridge hydraulic design procedure.

1. Determine the most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.
2. Determine design discharge from hydrologic studies or available data (City, Federal Emergency Management Agency (FEMA), US Army Corp of Engineers (USACE), TxDOT, or similar sources).
3. If available, obtain effective FEMA hydraulic backwater model. It is assumed that if a bridge is required instead of a culvert, the drainage area would exceed one square mile and could already be included in a FEMA study. If an effective FEMA model or other model is not available, a basic hydrologic model and backwater analysis for the stream must be prepared. The HEC-RAS computer model is routinely used to compute backwater water surface profiles.
4. Using USACE or FEMA guidelines, compute or duplicate an existing conditions water surface profile for the design storm(s). Compute a profile for the fully-developed watershed to use as a baseline for design of a new bridge/roadway crossing.
5. Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for preliminary sizing, use a normal-depth design procedure, or simply estimate a required trapezoidal opening.
6. Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard and any channelization upstream or downstream to transition the floodwaters through the proposed structure.
7. Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade, etc.). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.
8. Review the velocities and determine erosion control requirements downstream, through the structure, and upstream.
9. Finalize the design size and erosion control features, based on comparing the proposed model with the existing conditions profiles, impacts on other properties, FEMA guidelines, and Local Criteria.
10. Exceptions/Other Issues
    A. Conditional Letter of Map Amendment (CLOMR) may be needed for new crossings of streams studied by FEMA.
    B. If applicable, coordinate with USACE Regulatory Permit requirements.
    C. Evaluate the project with respect to iSWM policy regarding downstream impacts.
    D. Design should be for fully developed watershed conditions. If the available discharges are from FEMA existing conditions hydrology, the following options are available: (1) obtain new hydrology, (2) extrapolate fully-developed from existing data, or (3) variance from the local jurisdiction on design discharges
    E. Freeboard criteria may require an unusually expensive bridge or impracticable roadway elevation. A reasonable variance in criteria from the local jurisdiction may be available.

References

Texas Department of Transportation, March 2004. Hydraulic Design Manual. Available at http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd/@Generic__BookView.

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Section 4.4
Open Channel Design

4.4.1 Overview

4.4.1.1 Introduction

Open channel systems and their design are an integral part of storm water drainage design, particularly for development sites utilizing better site design practices and open channel structural controls. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, stone riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design criteria and methods, including the use of channel design nomographs.

4.4.1.2 Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see Section 1.4 and Chapter 3 for more details on using enhanced swales and grass channels for water quality purposes).

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities
- Standing or continuously flowing water
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of healthy vegetation.

If low flows are prevalent, a hard lined pilot channel may be needed, and it should be wide enough to accommodate maintenance equipment. Whether to allow pilot channels should be included in the local criteria section.

Flexible Linings – Rock riprap, including rubble and gabion baskets, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities

that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

4.4.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.4-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.4-1 Symbols and Definitions

Symbol	Definition	Units
α	Energy coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
delta d	Super-elevation of the water surface profile	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration of gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
w	Stone weight	lbs
y_c	Critical depth	ft

y_n	Normal depth	ft
z	Critical flow section factor	-

4.4.3 Design Criteria

4.4.3.1 General Criteria

The following criteria should be followed for open channel design:

- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
- Channel side slopes shall be stable throughout the entire length and the side slope shall depend on the channel material. A maximum of 2:1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum side slope of 3:1.
- Trapezoidal or parabolic cross sections are preferred over triangular shapes.
- For vegetative channels, design stability should be determined using low vegetative retardance conditions (Class D) and for design capacity higher vegetative retardance conditions (Class C) should be used.
- For vegetative channels, flow velocities within the channel should not exceed the maximum permissible velocities given in Tables 4.4-2 and 4.4-3.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.
- Open channel drainage systems are typically sized to handle a 25-year design storm. The 100-year design storm should be routed through the channel system to determine if the 100-year plus applicable building elevation restrictions are exceeded, structures are flooded, or flood damages increased.
- HEC-RAS is typically used to confirm the water surface profiles in open channels, and due to the complexity of hydraulic elements in roadside ditches (open channels), it should be used to evaluate water surface profiles in roadside ditches as well.

4.4.3.2 Velocity Limitations

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Recommended maximum velocity values for selected lining categories are presented in Table 4.4-2. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 4.4-3. Vegetative lining calculations are presented in Section 4.4.7 and stone riprap procedures are presented in Section 4.4.8.

Typically, local design limits the velocity to 15 fps in concrete lined channels. For gabions typical design velocities range from 10 fps for 6-inch mattresses up to 15 fps for 1-foot mattresses. Some manufacturers indicate that velocities of 20 fps are allowable for basket installations. No specific velocity limit is appropriate for rock riprap. The design of stable riprap lining depends upon the intersection of the velocity (local boundary shear) and the size and gradation of the riprap material. In general, velocity limitations should be set by the local jurisdiction.

4.4.4 Manning's n Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and stone riprap linings are given in Table 4.4-4. Recommended values for vegetative linings should be determined using Figure 4.4-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 4.4-6). Figure 4.4-1 is used iteratively as described in Section 4.4.6. Recommended Manning's values for natural channels that are either excavated or dredged, and natural are given in Table 4.4-2. For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959. Some of these values are given in Table 4.4-2 below.

Table 4.4-2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
MINOR NATURAL STREAMS		
Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	3 to 6
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	3 to 6
3. Some weeds, light brush on banks	0.035	3 to 6
4. Some weeds, heavy brush on banks	0.050	3 to 6
5. Some weeds, dense willows on banks	0.060	3 to 6
For trees within channels with branches submerged at high stage, increase above values by	0.010	
Irregular section with pools, slight channel meander, increase above values by	0.010	
Floodplain – Pasture		
1. Short grass	0.030	3 to 6
2. Tall grass	0.035	3 to 6
Floodplain – Cultivated Areas		
1. No crop	0.030	3 to 6
2. Mature row crops	0.035	3 to 6
3. Mature field crops	0.040	3 to 6
Floodplain – Uncleared		
1. Heavy weeds scattered brush	0.050	3 to 6
2. Wooded	0.120	3 to 6
MAJOR NATURAL STREAMS		
Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or	Range from 0.028 to 0.060	3 to 6

Table 4.4-2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
vegetation on banks. Values of "n" for larger streams of mostly regular sections, with no boulders or brush		
UNLINED VEGETATED CHANNELS		
Clays (Bermuda Grass)	0.035	5 to 6
Sandy and Silty Soils (Bermuda Grass)	0.035	3 to 5
UNLINED NON-VEGETATED CHANNELS		
Sandy Soils	0.030	1.5 to 2.5
Silts	0.030	0.7 to 1.5
Sandy Silts	0.030	2.5 to 3.0
Clays	0.030	3.0 to 5.0
Coarse Gravels	0.030	5.0 to 6.0
Shale	0.030	6.0 to 10.0
Rock	0.025	15

Table 4.4-3 Maximum Velocities for Vegetative Channel Linings

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda grass	0-5	6
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	5-10	5
	>10	4
Grass mixture	0-5[1]	4
	5-10	3
Sericea lespedeza, Weeping lovegrass, Alfalfa	0-5[4]	3
Annuals[5]	0-5	3
Sod		4
Lapped sod		5

[1] Do not use on slopes steeper than 10% except for side-slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stands can be maintained.
[3] Mixtures of Tall Fescue, Bahia, and/or Bermuda
[4] Do not use on slopes steeper than 5% except for side-slope in combination channel.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Source: Manual for Erosion and Sediment Control in Georgia, 1996

Table 4.4-4 Manning's Roughness Coefficients (n) for Artificial Channels

		Depth Ranges		
Category	Lining Type	0-0.5 ft	0.5-2.0 ft	> 2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	–	0.078	0.040

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

Source: HEC-15, 1988.

When designing open channels, the usual choice of Manning's roughness coefficients may be found in Table 4.4-5. The local jurisdiction may choose to vary from these values.

Table 4.4-5 Manning's Roughness Coefficients for Design

Lining Type	Manning's n	Comments
Grass Lined	0.035	Use for velocity check.
	0.050	Use for channel capacity check (freeboard check)
Concrete Lined	0.015	
Gabions	0.030	
Rock Riprap	0.040	$n = 0.0395 d_{50}^{1/6}$ where d_{50} is the stone size of which 50% of the sample is smaller
Grouted Riprap	0.028	FWHA

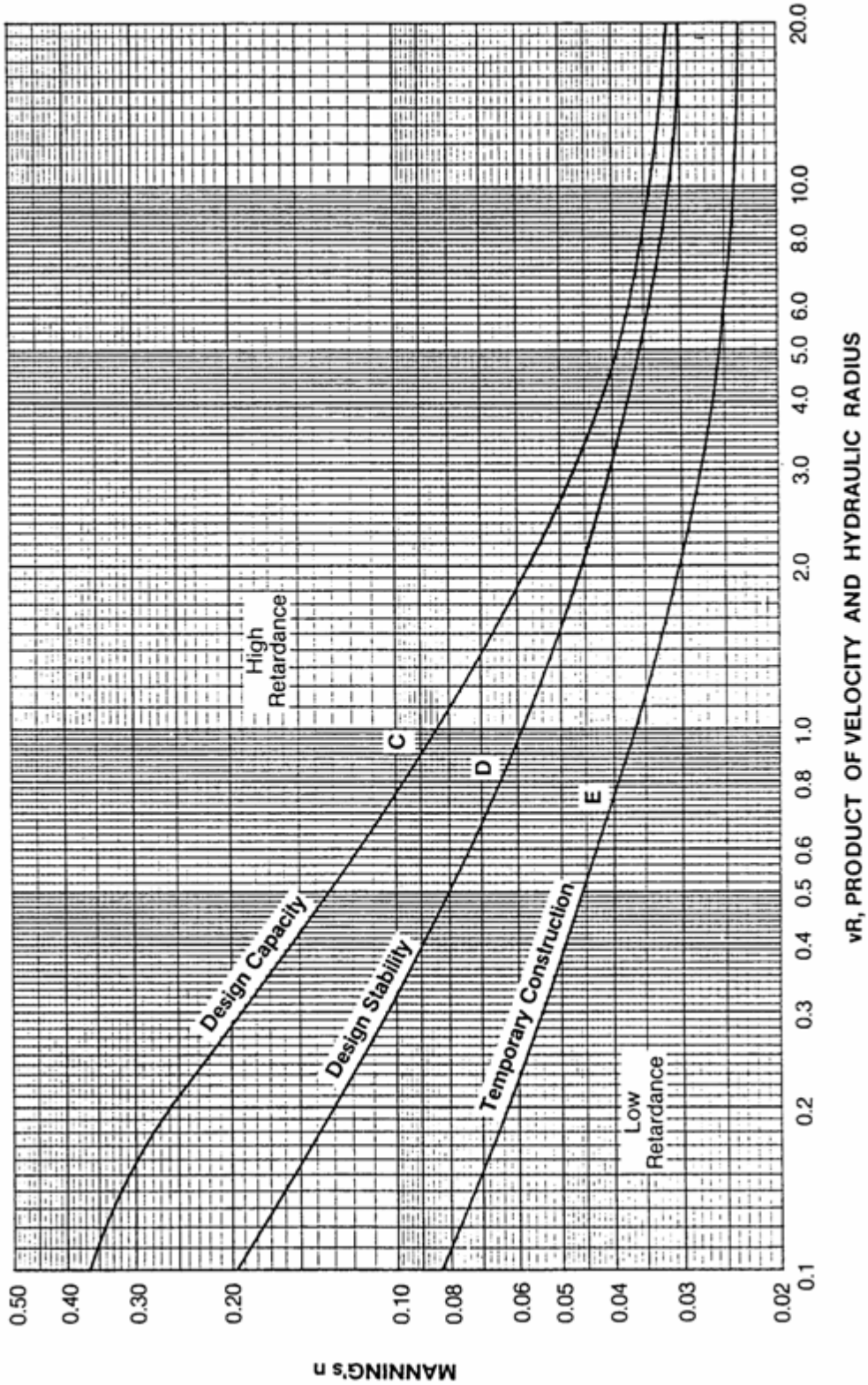


Figure 4.4-1 Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 4.4-6 Classification of Vegetal Covers as to Degrees of Retardance

Retardance	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36")
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture	
	Little bluestem, bluestem, blue gamma other short and long stem Midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24")
	Laspedeza sericea	Good stand, not woody, tall (average 19")
	Alfalfa	Good stand, uncut (average 11")
	Weeping lovegrass	Good stand, unmowed (average 13")
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13")
C	Crabgrass	Fair stand, uncut (10 – 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture:	
	summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 – 8 ")
	Centipede grass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 – 12")
D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3 – 6")
	Grass-legume mixture:	
	fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 – 5")
	Lespedeza serices	After cutting to 2" (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.
Source: HEC-15, 1988

4.4.5 Uniform Flow Calculations

4.4.5.1 Design Charts

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, trapezoidal, and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. Examples of these charts and instructions for their use are given in subsection 4.4.12.

4.4.5.2 Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = (1.49/n)\, R^{2/3}\, S^{1/2} \qquad (4.4.1)$$

$$Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2} \qquad (4.4.2)$$

$$S = [Q_n/(1.49\, A\, R^{2/3})]^2 \qquad (4.4.3)$$

where:

- v = average channel velocity (ft/s)
- Q = discharge rate for design conditions (cfs)
- n = Manning's roughness coefficient
- A = cross-sectional area (ft^2)
- R = hydraulic radius A/P (ft)
- P = wetted perimeter (ft)
- S = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

For a more comprehensive discussion of open channel theory and design, see the reference USACE, 1991/1994.

4.4.5.3 Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

4.4.5.4 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from equation 4.4.2. The slope can be calculated using equation 4.4.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 4.4-2 and 4.4-3. Figure 4.4-2 provides a general solution for the velocity form of Manning's Equation, while Figure 4.4-3 provides a solution of Manning's Equation for trapezoidal channels.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 4.4-2:

Step 1 Determine open channel data, including slope in ft/ft, hydraulic radius in ft, and Manning's n value.

Step 2 Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

Step 3 Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

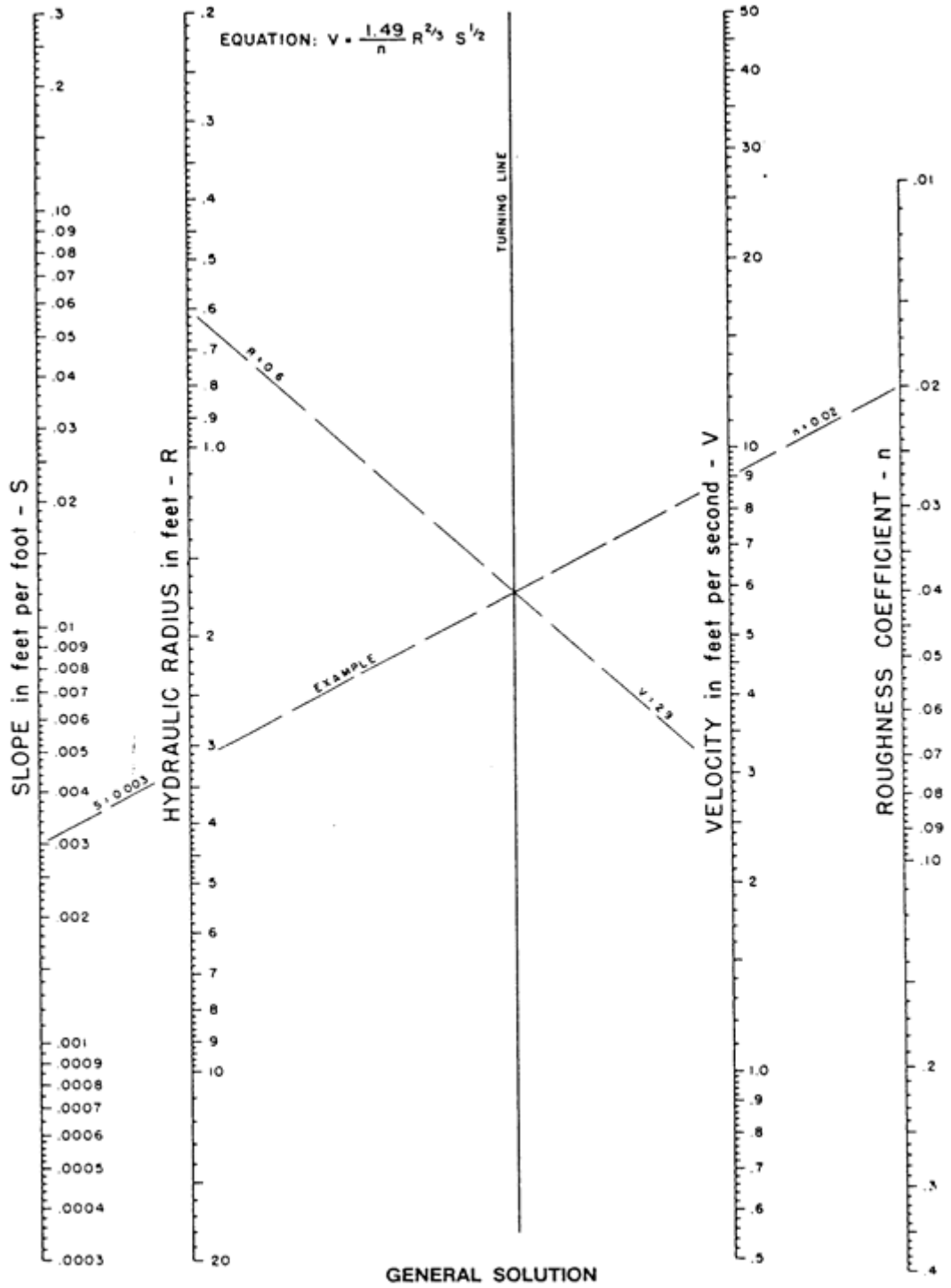
Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 4.4-3 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

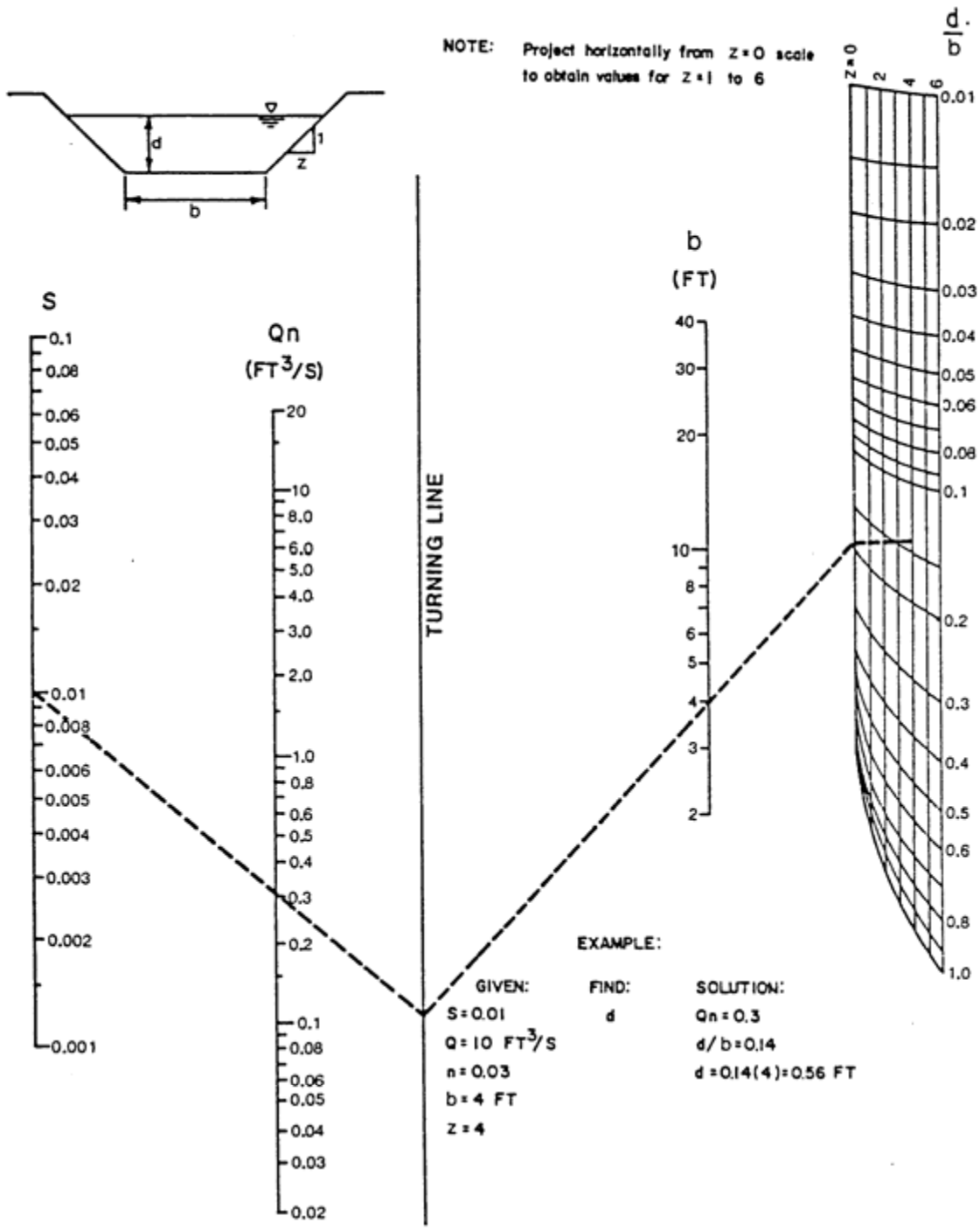
1. Given Q, find d.
   a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.
   b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.
   c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.
   d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.
2. Given d, find Q

   Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.
   Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.
   Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.
   Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 4.4-2 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 4.4-3 Solution of Manning's Equation for Trapezoidal Channels

4.4.5.5 Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as:

$$AR^{2/3} = (Qn)/(1.49\ S^{1/2}) \qquad (4.4.4)$$

where:

A = cross-sectional area (ft)
R = hydraulic radius (ft)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
S = slope of the energy grade line (ft/ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of equation 4.4.4 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 4.4-4 for trapezoidal channels. Computer programs are also available for these calculations.

Step 1 Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

Step 2 Calculate the trapezoidal conveyance factor using the equation:

$$K_T = (Qn)/(b^{8/3}S^{1/2}) \qquad (4.4.5)$$

where:

K_T = trapezoidal open channel conveyance factor
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
b = bottom width (ft)
S = slope of the energy grade line (ft/ft)

Step 3 Enter the x-axis of Figure 4.4-4 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

Step 4 From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

Step 5 Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

Note: If bends are considered, refer to equation 4.4.11

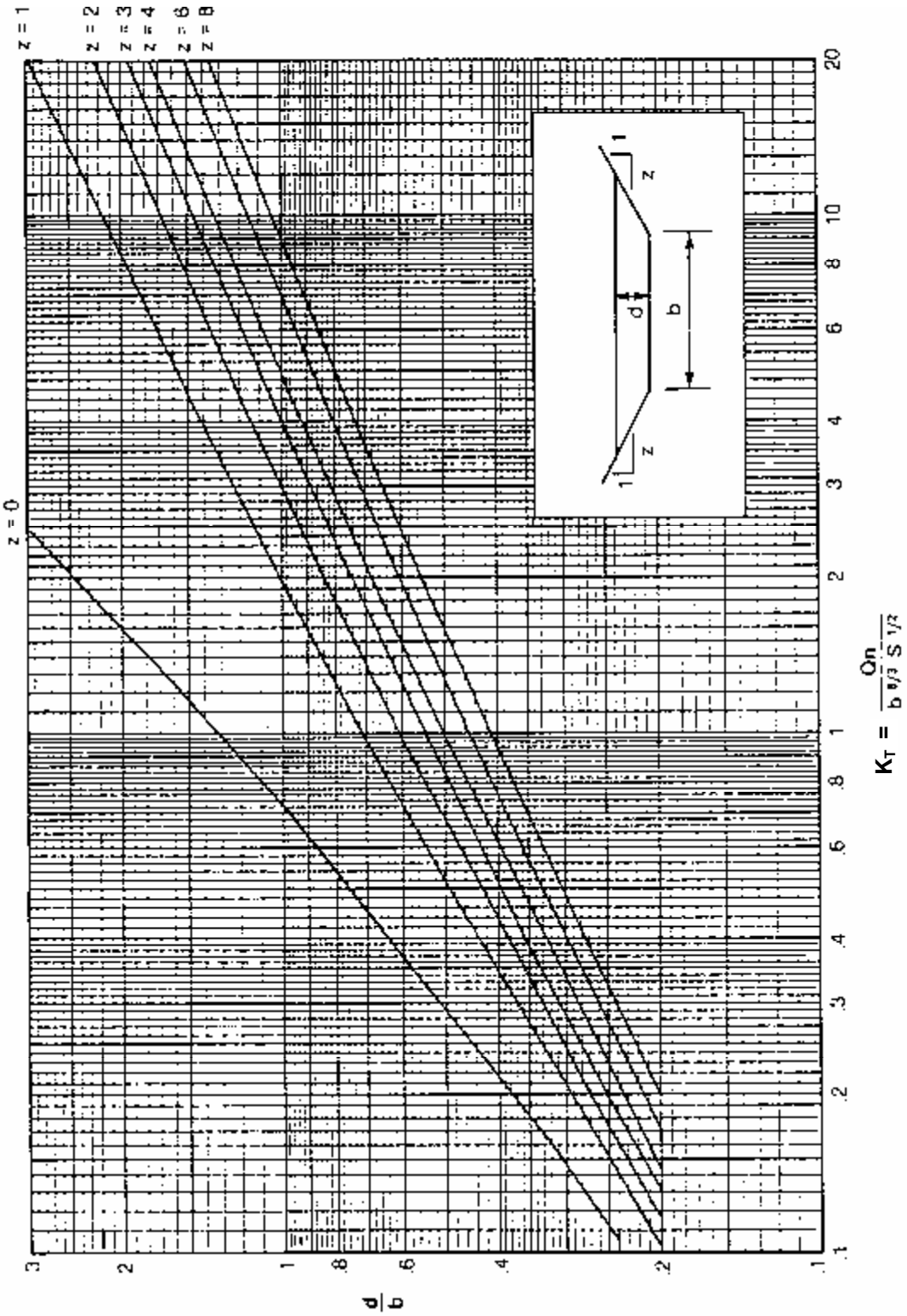


Figure 4.4-4 Trapezoidal Channel Capacity Chart
(Source: Nashville Storm Water Management Manual, 1988)

4.4.6 Critical Flow Calculations

4.4.6.1 Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities. Hydraulic jumps are possible under these conditions and consideration should be given to stabilizing the channel.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$Q^2/g = A^3/T \qquad (4.4.6)$$

where:

- Q = discharge rate for design conditions (cfs)
- g = acceleration due to gravity (32.2 ft/sec^2)
- A = cross-sectional area (ft^2)
- T = top width of water surface (ft)

Note: A trial and error procedure is needed to solve equation 4.4-6.

4.4.6.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 4.4-7) or section factors (as presented in Figure 4.4-5) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$Z = Q/(g^{0.5}) \qquad (4.4.7)$$

where:

- Z = critical flow section factor
- Q = discharge rate for design conditions (cfs)
- g = acceleration due to gravity (32.3 ft/sec^2)

The following guidelines are given for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
3. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
4. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
5. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

Note: The head is the height of water above any point, plane, or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant ($V^2/2g$).

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/(gA/T)^{0.5} \quad (4.4.8)$$

where:

- Fr = Froude number (dimensionless)
- v = velocity of flow (ft/s)
- g = acceleration of gravity (32.2 ft/sec^2)
- A = cross-sectional area of flow (ft^2)
- T = top width of flow (ft)

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 4.4-7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75b1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	N/A
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	N/A

where:

- d_c = critical depth (ft)
- Q = design discharge (cfs)
- g = acceleration due to gravity (32.3 ft/s^2)
- b = bottom width of channel (ft)
- z = side slopes of a channel (horizontal to vertical)
- D = diameter of circular conduit (ft)
- A = cross-sectional area of flow (ft^2)
- T = top width of water surface (ft)

[1] See Figure 4.4-5 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

If the water surface profile in a channel transitions from supercritical flow to subcritical flow, a hydraulic jump must occur. The location of the hydraulic jump and its sequent depth are critical to proper design of free flow conveyances. To determine the location of a hydraulic jump, the standard step method is used to compute the water surface profile and specific force (momentum principle) and specific energy relationships are used. For computational methods refer to Chow, 1959, TxDOT, 2002, and Mays, 1999. The HEC-RAS computer program can be used to compute water surface profiles for both subcritical and supercritical flow regimes.

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2+1}$	$\frac{bd + zd^2}{b + 2d\sqrt{z^2+1}}$	$b + 2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
Rectangle	bd	$b + 2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2} zd^{2.5}$
Parabola	$\frac{2}{3} dT$	$T + \frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\, Td^{1.5}$
Circle - < 1/2 full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180} - \sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180} - \sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
Circle - > 1/2 full [3]	$\frac{D^2}{8}\left(2\pi - \frac{\pi\theta}{180} + \sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi - \frac{\pi\theta}{180} + \sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956)

Figure 4.4-5 Open Channel Geometric Relationships for Various Cross Sections

4.4.7 Vegetative Design

4.4.7.1 Introduction

A two-part procedure is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 4.4-6. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 4.4-6. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10%, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

4.4.7.2 Design Stability

The following are the steps for design stability calculations:

Step 1 Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

Step 2 Use Table 4.4-3 to assign a maximum velocity, v_m based on vegetation type and slope range.

Step 3 Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 4.4-1. Use retardance Class D for permanent vegetation and E for temporary construction.

Step 4 Calculate the hydraulic radius using the equation:

$$R = (vR)/v_m \qquad (4.4.9)$$

where:

R = hydraulic radius of flow (ft)

vR = value obtained from Figure 4.4-1 in Step 3

v_m = maximum velocity from Step 2 (ft/s)

Step 5 Use the following form of Manning's Equation to calculate the value of vR:

$$vR = (1.49\ R^{5/3}\ S^{1/2})/n \qquad (4.4.10)$$

where:

vR = calculated value of vR product

R = hydraulic radius value from Step 4 (ft)

S = channel bottom slope (ft/ft)

n = Manning's n value assumed in Step 3

Step 6 Compare the vR product value obtained in Step 5 to the value obtained from Figure 4.4-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

Step 7 For trapezoidal channels, find the flow depth using Figures 4.4-3 or 4.4-4, as described in Section 4.4.4.4. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 4.4.4.5.

Step 8 If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 4.4-6. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

4.4.7.3 Design Capacity

The following are the steps for design capacity calculations:

Step 1 Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 4.4-5 for equations).

Step 2 Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

Step 3 Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

Step 4 Use Figure 4.4-1 to find a Manning's n value for retardance Class C based on the vR value from Step 3.

Step 5 Use Manning's Equation (equation 4.4.1) or Figure 4.4-2 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

Step 6 Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

Step 7 Add an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

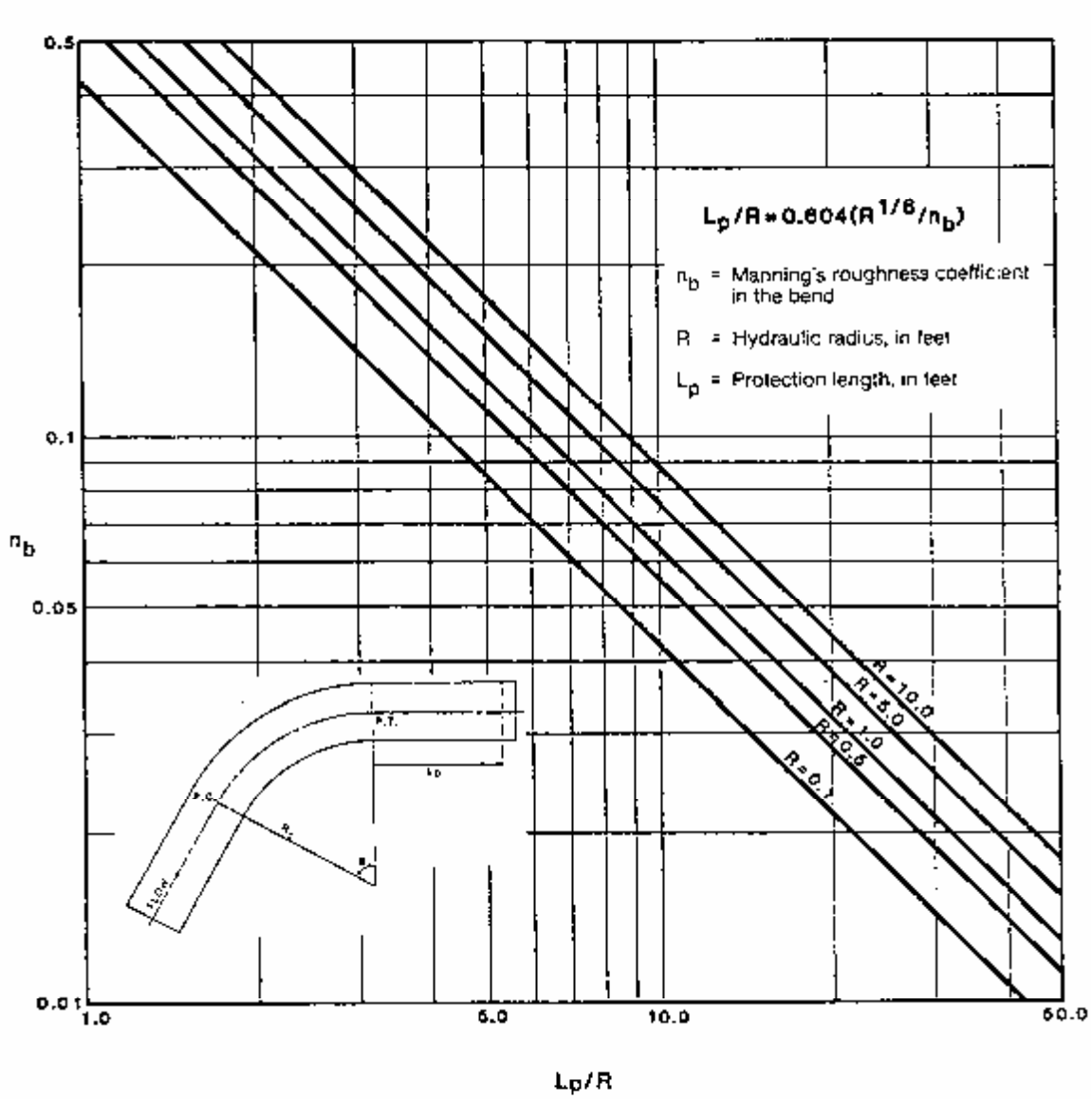
Step 8 If bends are considered, calculate super-elevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2 T)/(g R_c) \qquad (4.4.11)$$

where:

Δd = super-elevation of the water surface profile due to the bend (ft)
v = average velocity from Step 6 (ft/s)
T = top width of flow (ft)
g = acceleration of gravity (32.2 ft/sec^2)
R_c = mean radius of the bend (ft)

Note: Add freeboard consistent with the calculated Δd.



Reference. USDOT, FHWA, HEC-15 (1986).

Figure 4.4-6 Protection Length, Lp, Downstream of Channel Bend

4.4.8 Stone Riprap Design

4.4.8.1 Introduction

A number of agencies and researchers have studied and developed empirical equations to estimate the required size of rock riprap to resist various hydraulic conditions, including the U.S. Army Corps of Engineers (USACE), Natural Resource Conservation Service (NRCS), and the Federal Highway Administration (FHWA). As with all empirical equations based on the results of laboratory experiments, they must be used with an understanding of the range of data on which they are based.

Sections 4.4.8.2 and 4.4.8.3 give design guidance for designing stone riprap for open channels. Section 4.4.8.4 gives design guidance for designing stone riprap for culvert outfall protection. Section 4.7.4 gives additional guidance on the design of riprap aprons for erosion protection at outfalls, and Section 4.7.5 gives guidance on the design of riprap basins for energy dissipation.

4.4.8.2 Method #1: Maynord & Reese

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

- Minimum riprap thickness equal to d_{100}
- The value of d_{85}/d_{15} less than 4.6
- Froude number less than 1.2
- Side slopes up to 2:1
- A safety factor of 1.2
- Maximum velocity less than 18 feet per second
- If significant turbulence is caused by boundary irregularities, such as vertical drops, obstructions, or structures, this procedure is not applicable.

<u>Procedure</u>

Following are the steps in the procedure for riprap design using the method by Maynord & Reese:

Step 1 Determine the average velocity in the main channel for the design condition. Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395\,(d_{50})^{1/6} \qquad (4.4.12)$$

where:

n = Manning's roughness coefficient for stone riprap

d_{50} = diameter of stone for which 50%, by weight, of the gradation is finer (ft)

Step 2 If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 4.4-7 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

Step 3 If the specific weight of the stone varies significantly from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 4.4-8.

Step 4 Determine the required minimum d_{30} value from Figure 4.4-9, or from the equation:

$$d_{30}/D = 0.193\,Fr^{2.5} \qquad (4.4.13)$$

where:

d_{30} = diameter of stone for which 30%, by weight, of the gradation is finer (ft)

D = depth of flow above stone (ft)

Fr = Froude number (see equation 4.4.8), dimensionless

v = mean velocity above the stone (ft/s)

g = acceleration of gravity (32.2 ft/sec)

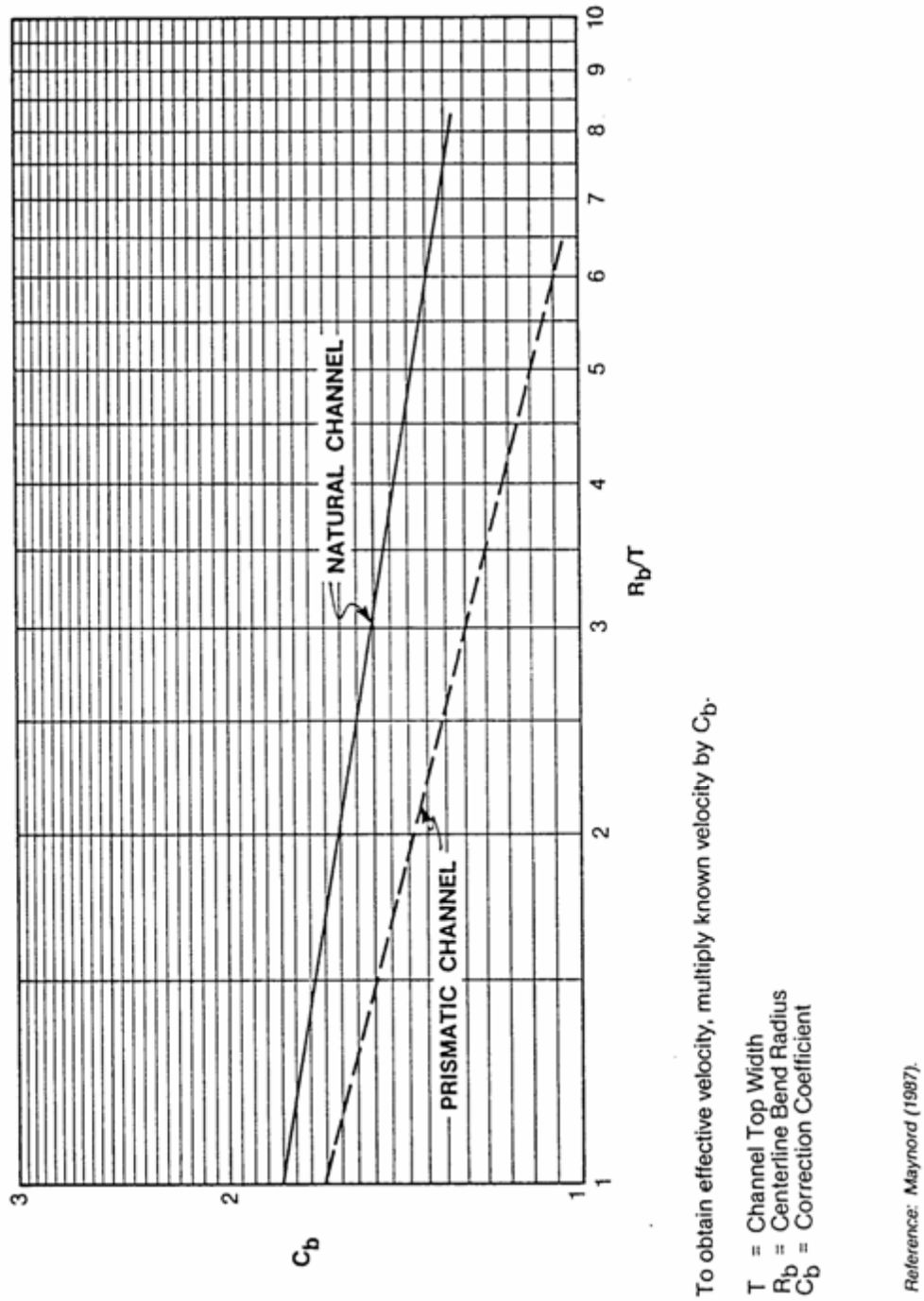
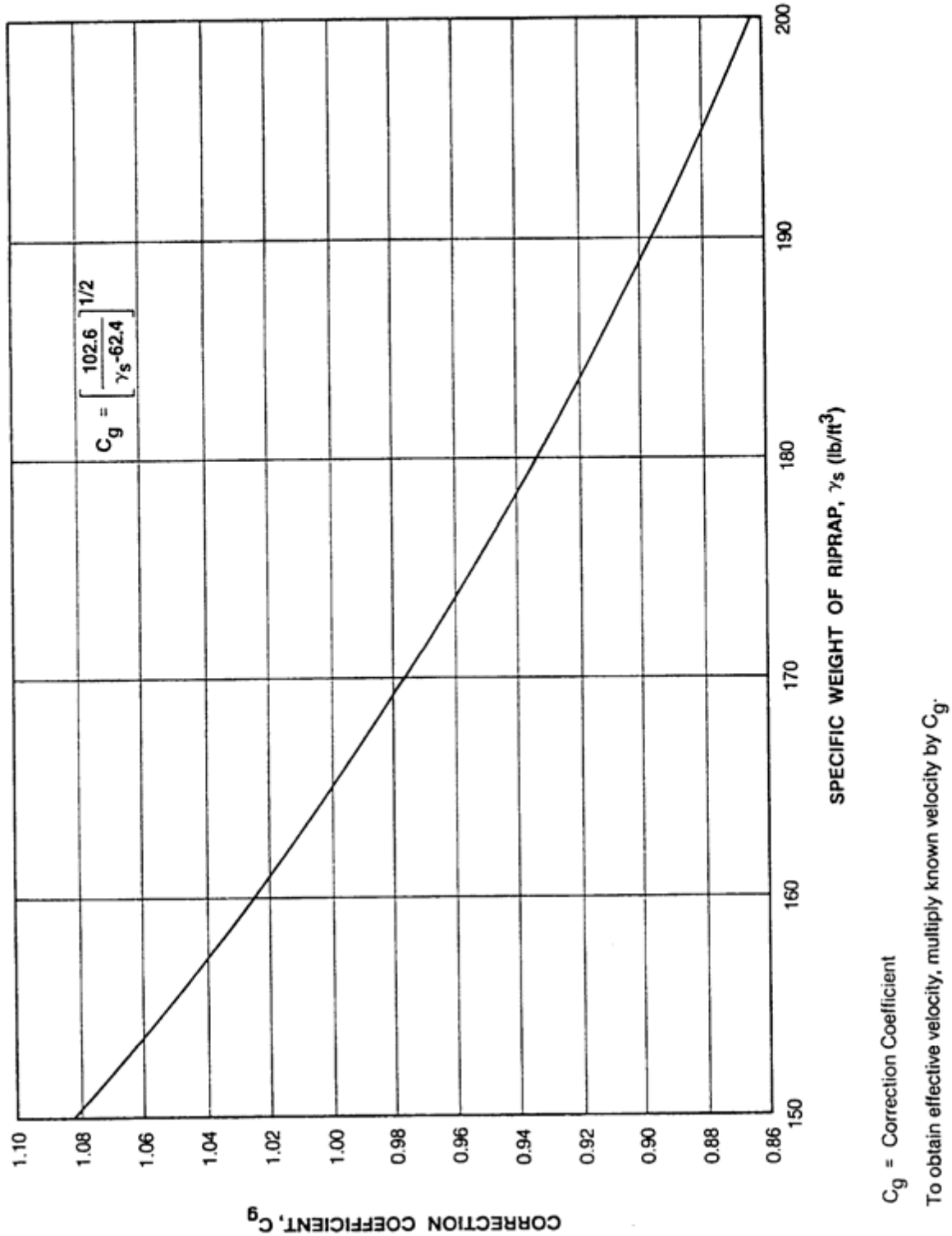


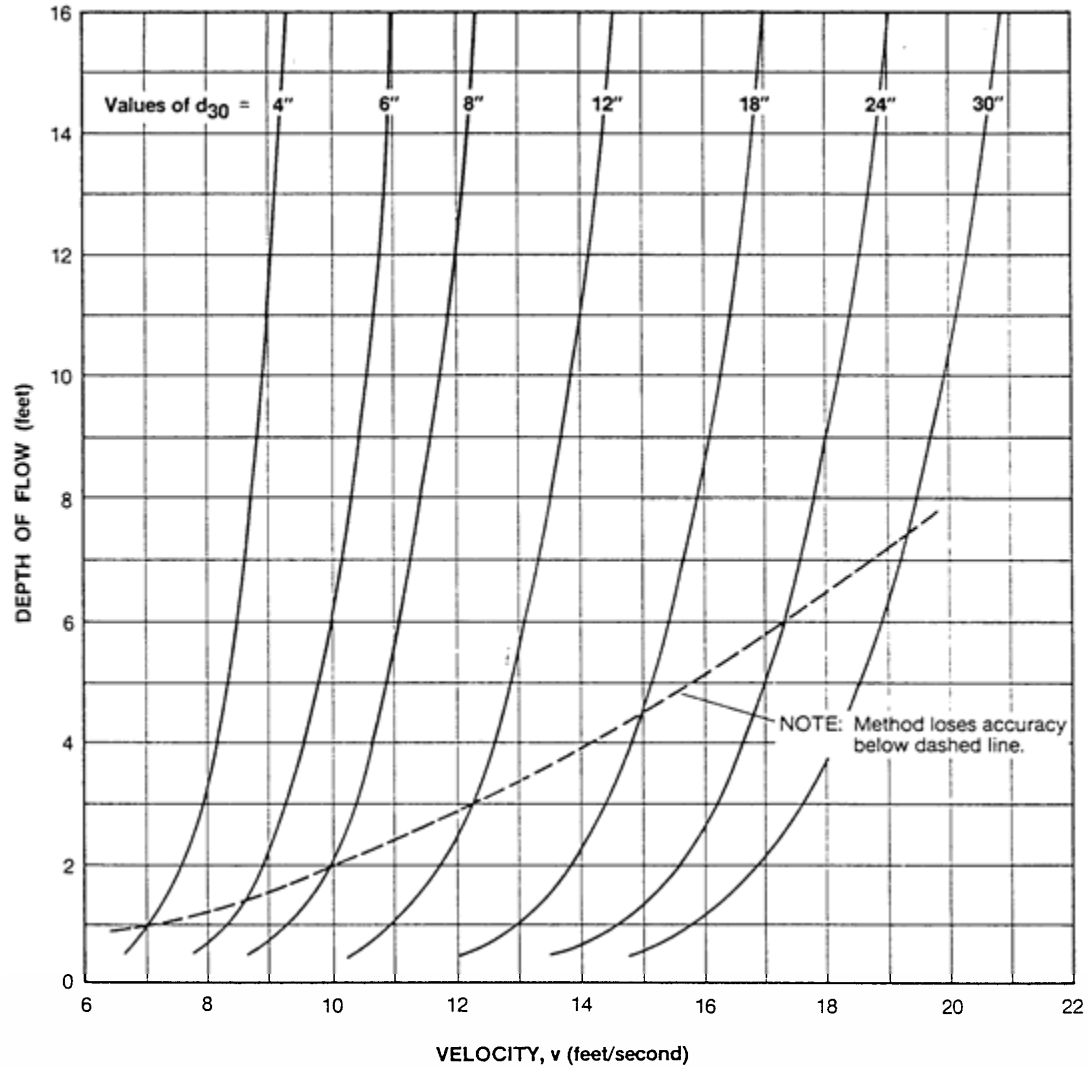
Figure 4.4-7 Riprap Lining Bend Correction Coefficient



C_g = Correction Coefficient

To obtain effective velocity, multiply known velocity by C_g.

Figure 4.4-8 Riprap Lining Specific Weight Correction Coefficient
(Source: Nashville Storm Water Management Manual, 1988)



Reference: Reese (1988).

Figure 4.4-9 Riprap Lining d_{30} Stone Size – Function of Mean Velocity and Depth

Step 5 Determine available riprap gradations. A well graded riprap is required. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$W = 0.5236\ SW_s\ d^3 \qquad (4.4.14)$$

where:

W = stone weight (lbs)

d = selected stone diameter (ft)

SW_s = specific weight of stone (lbs/ft^3)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50% for underwater placement.

Step 6 If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 4.4-10 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

Step 7 Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

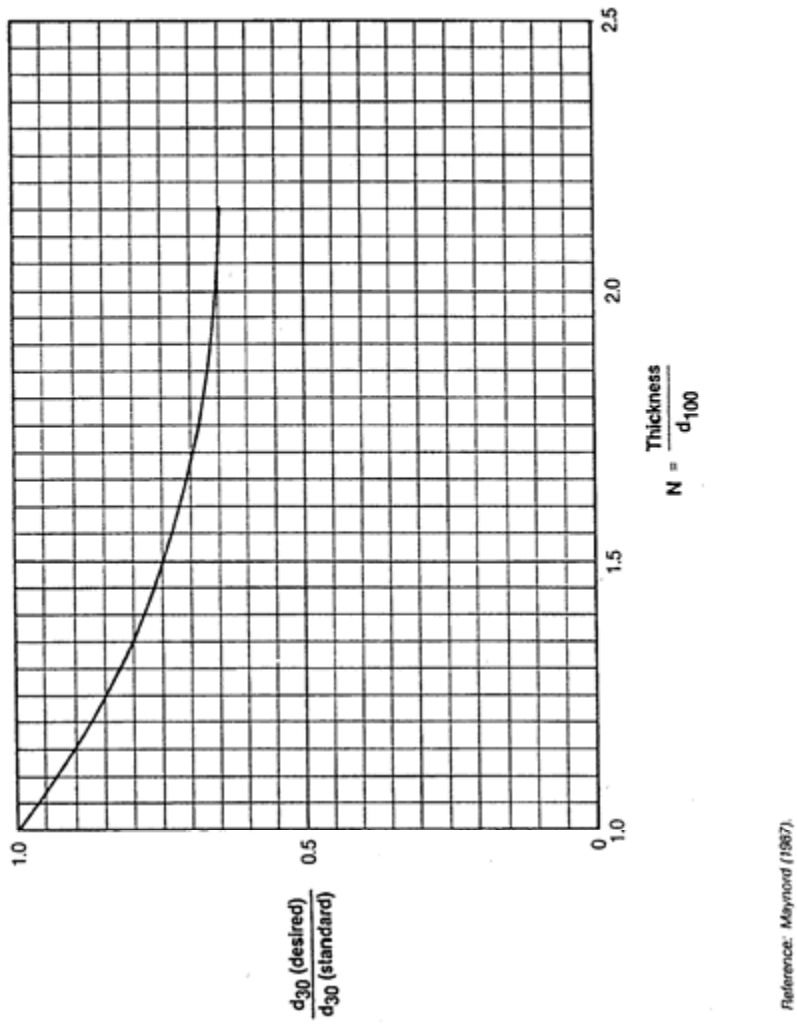


Figure 4.4-10 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3
(Source: Maynord, 1987)

4.4.8.3 Method #2: Gregory

The following procedure is based on excerpts from a paper prepared by Garry Gregory (June, 1987) and has been widely used in the Dallas-Fort Worth area for riprap design.

Procedure

Following are the steps in the procedure for riprap design using the method by Gregory:

Step 1 Calculate the boundary shear (tractive stress or tractive force) by:

$$T_o = \Upsilon_w RS \qquad (4.4.15)$$

where:

T_o = average tractive stress on channel bottom (lb/ft^2)
Υ_w = unit weight of water (62.4 lb/ft^3)
R = hydraulic radius of channel (ft)
S = slope of energy gradient (ft/ft)

$$T_o' = T_o((1-\sin^2\Phi)/\sin^2\Theta)^{0.5} \qquad (4.4.16)$$

where:

T_o' = average tractive stress on channel side slopes (lb/ft^2)
Φ = angle of side slope with the horizontal
Θ = angle of repose of riprap (approximately 40°)

The greater value of T_o or T_o' governs.

Step 2 Determine the tractive stress in a bend in the channel by:

$$T_b = 3.15T(r/w)^{-0.5} \qquad (4.4.17)$$

where:

T_b = local tractive stress in the bend (lb/ft^2)
T = the greater of T_o or T_o' from equations 4.4.15 and 4.4.16
r = center-line radius of the bend (ft)
w = water surface width at upstream end of bend (ft)

Step 3 Determine D_{50} size of riprap stone (size at which 50% of the gradation is finer weight) from:

$$D_{50} = T/0.04(\Upsilon_s-\Upsilon_w) \qquad (4.4.18)$$

where:

D_{50} = required average size of riprap stone (ft)
T = the greater of T_o or T_o' from equations 4.4.15 and 4.4.16
Υ_s = saturated surface dry (SSD) unit weight of stone (lb/ft^3)
Υ_w = unit weight of water (62.4 lb/ft^3)

Step 4 Select minimum riprap thickness required from grain size curves in Figures 4.4-11 through 4.4-16. Select from smaller side of band at 50% finer gradation.

Step 5 Select riprap gradations table (Figures 4.4-17 and 4.4-18) based on riprap thickness selected in Step 4.

Step 6 Select bedding thickness from grain size curves in Figures 4.4-11 through 4.4-16, which was used to select the riprap thickness in Step 4. Note: The bedding thicknesses included in Figures 4.4-11 through 4.4-16 are based on using a properly designed geotextile underneath

the bedding. If a geotextile is not used, the bedding thickness must be increased to a minimum of 9 inches for 24 inch and 30 inch riprap and a minimum of 12 inches for the 36 inch riprap.

Step 7 To provide stability in the riprap layer the riprap gradations should meet the following criteria for GRADATION INDEX:

$$\textbf{GRADATION INDEX: } [D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5 \qquad (4.4.19)$$

where: D_{85}, D_{50}, and D_{15} are the riprap grain sizes (mm) of which 85%, 50%, and 15% respectively are finer by weight.

Step 8 To provide stability of the bedding layer the bedding should meet the following filter criteria with respect to the riprap:

$$D_{15}/d_{85} < 5 < D_{15}/d_{15} < 40 \qquad (4.4.20)$$

$$D_{50}/d_{50} < 40 \qquad (4.4.21)$$

where: D refers to riprap sizes, and d refers to bedding sizes, both in mm.

Step 9 The geotextile underneath the bedding should be designed as a filter to the soil.

Step 10 Typical riprap design sections are shown in Figures 4.4-19 and 4.4-20, from the USACE publication EM1110-2-160.

4.4.8.4 Culvert Outfall Protection

The following procedure is used to design riprap for protection at culvert outfalls.

Step 1 Determine D_{50} size of riprap determined from:

$$D_{50} = \sqrt{V}/[1.8\sqrt{(2g(\Upsilon_s-\Upsilon_w)/\Upsilon_w)}] \qquad (4.4.22)$$

Step 2 Select riprap and bedding from Figures 4.4-11 through 4.4-16 using D_{50} from equation 4.4.22.

Step 3 Select gradations from tables in Figures 4.4-17 and 4.4-18

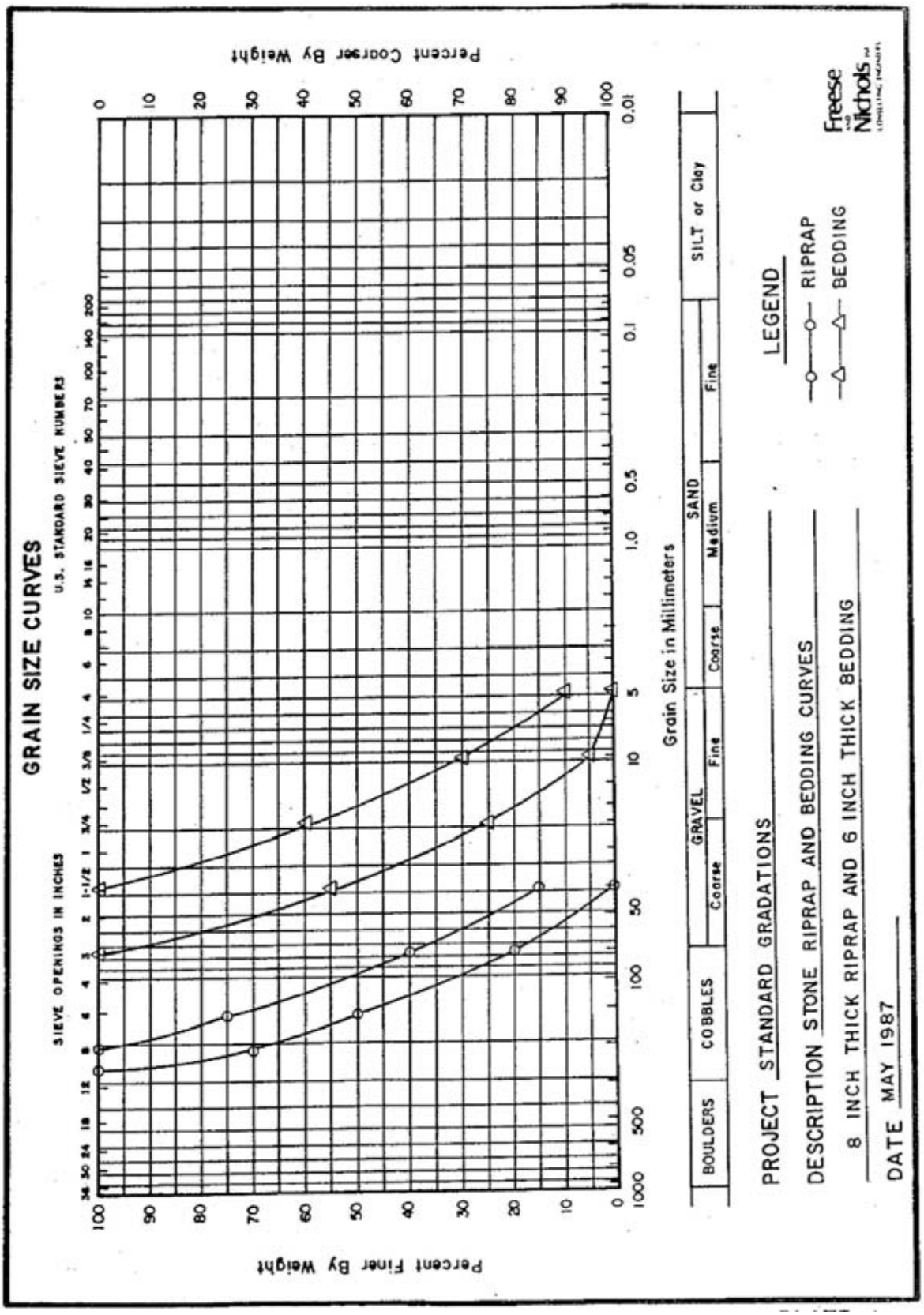


Figure 4.4-11 Grain Size Curve for 8" Riprap and 6" Bedding

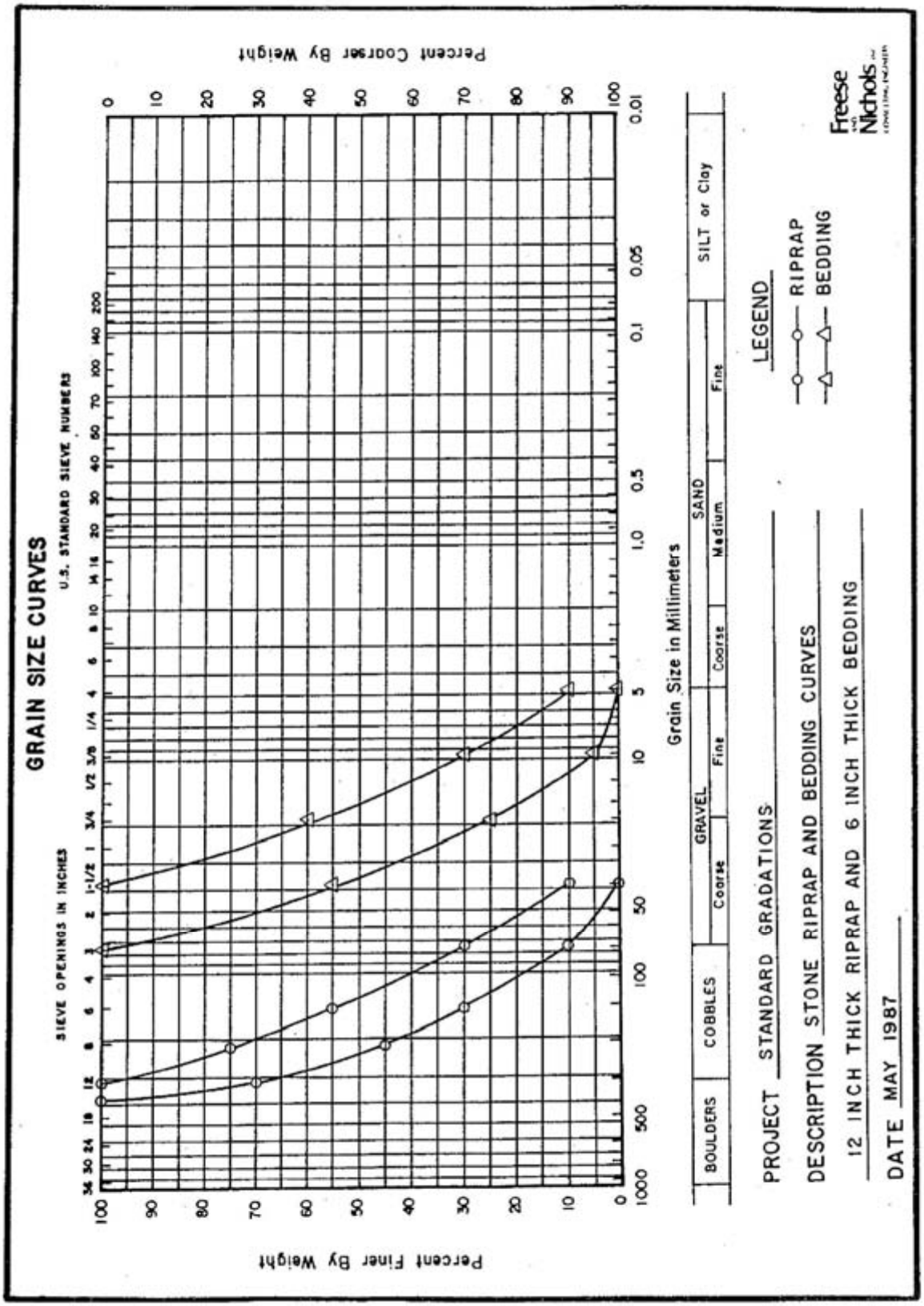


Figure 4.4-12 Grain Size Curve for 12" Riprap and 6" Bedding

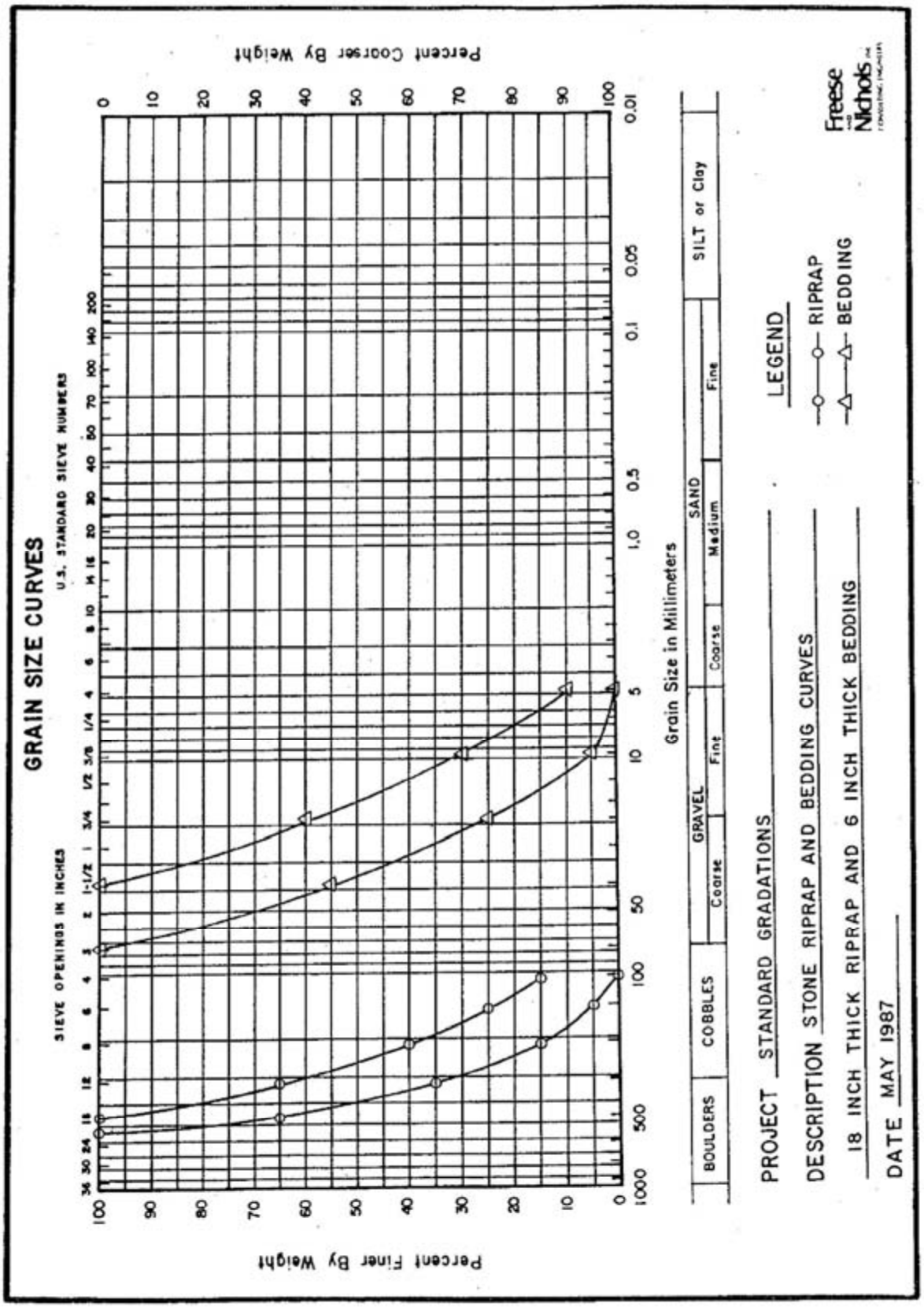


Figure 4.4-13 Grain Size Curve for 18" Riprap and 6" Bedding

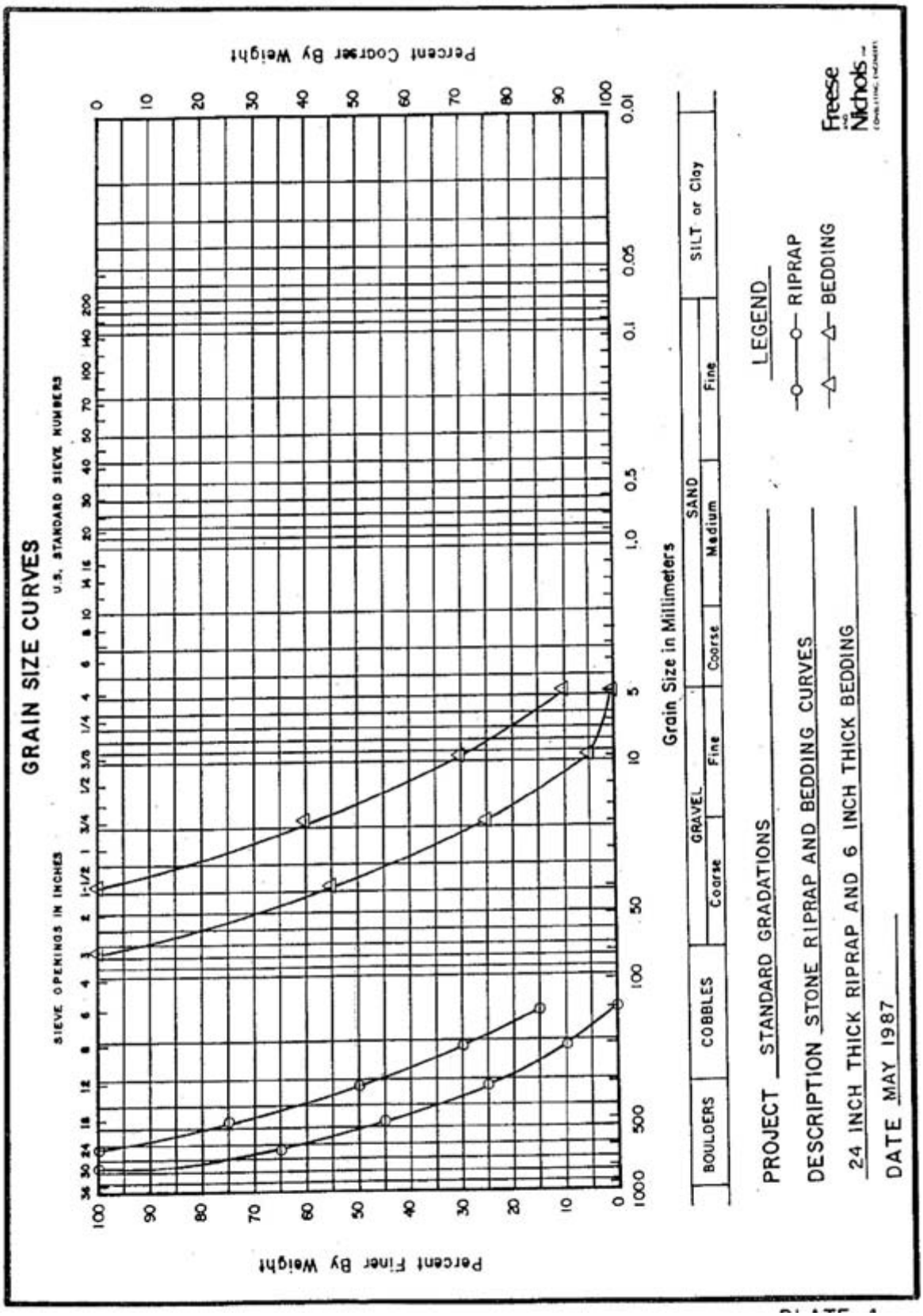


Figure 4.4-14 Grain Size Curve for 24" Riprap and 6" Bedding

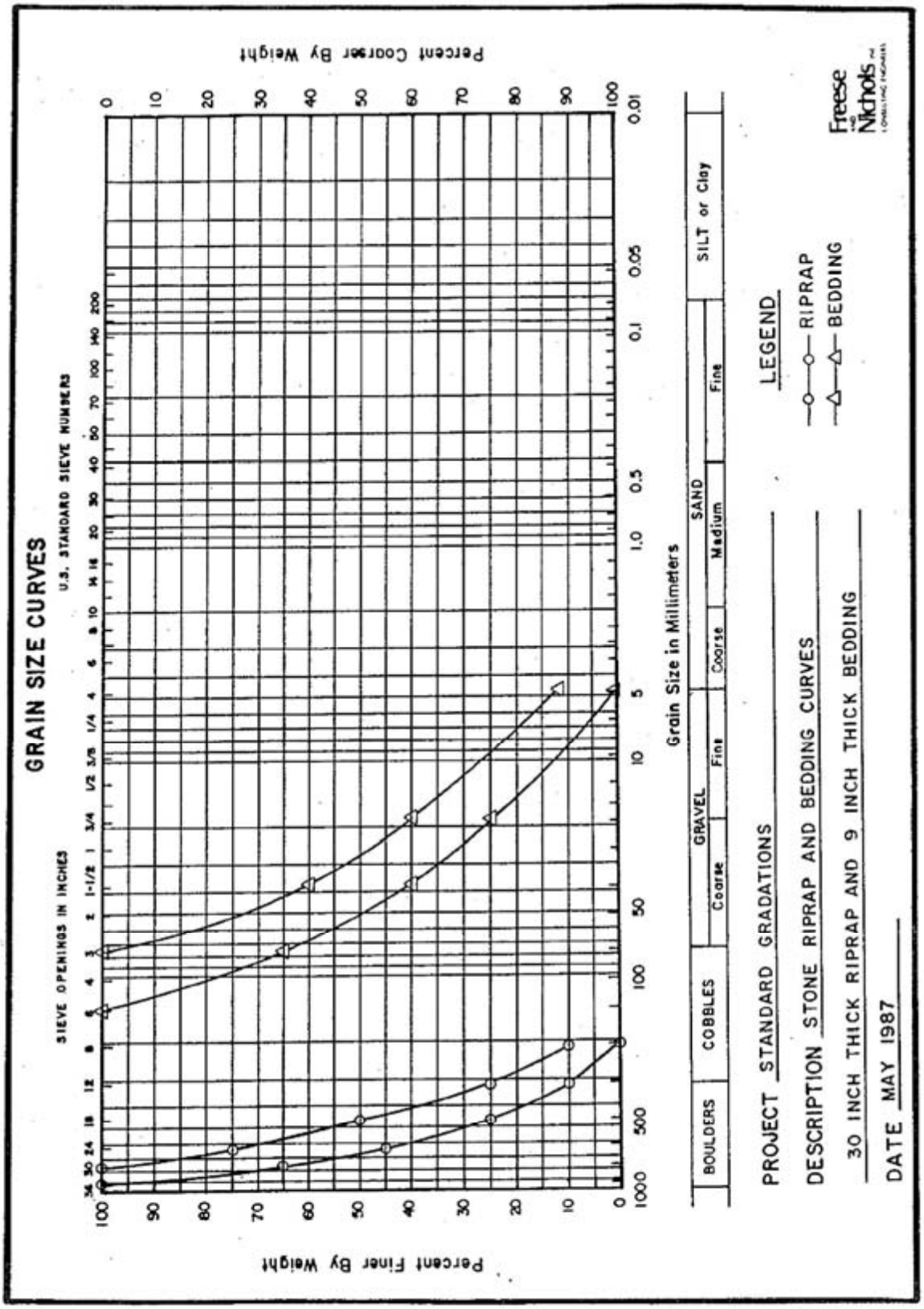


Figure 4.4-15 Grain Size Curve for 30" Riprap and 9" Bedding

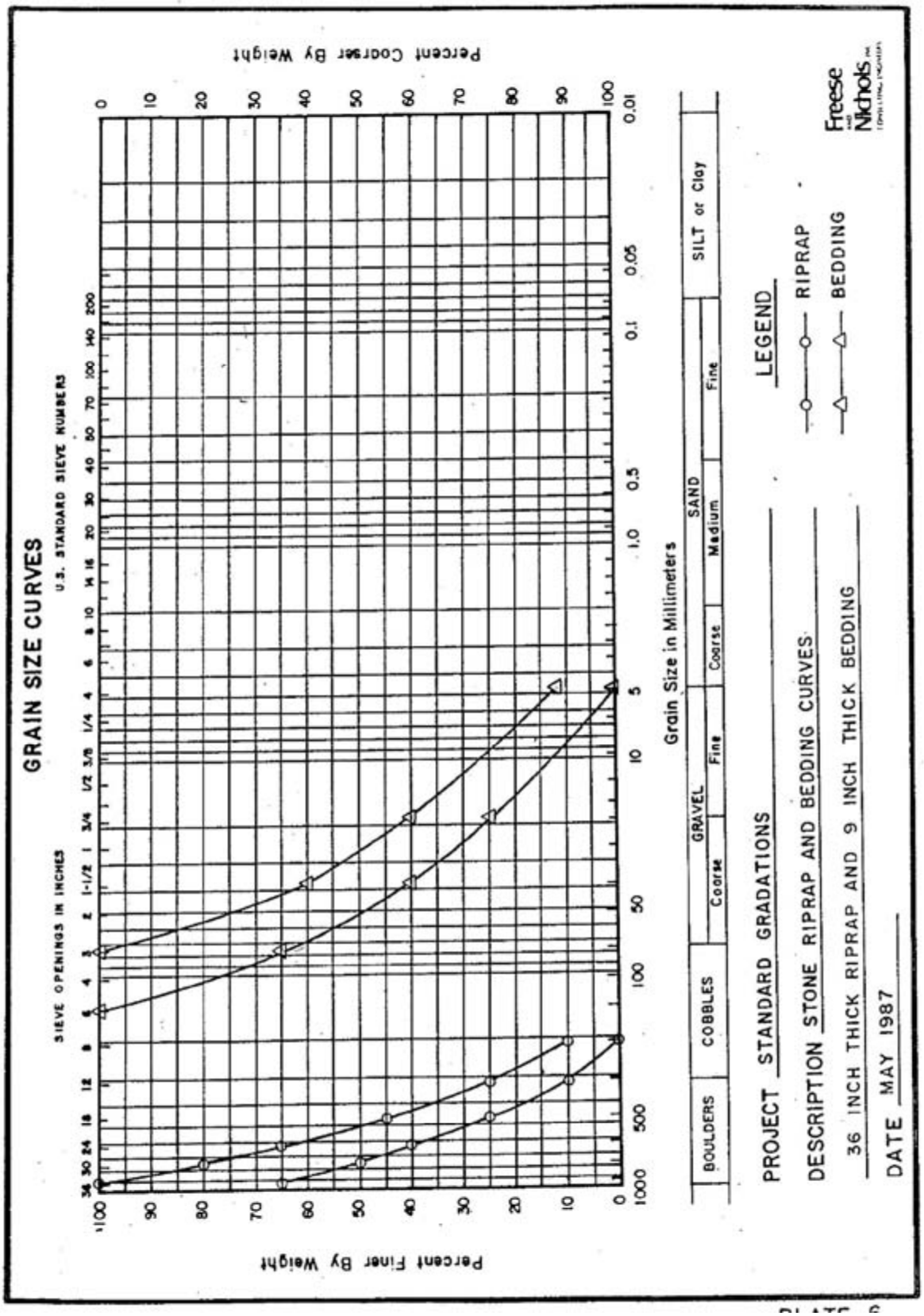


Figure 4.4-16 Grain Size Curve for 36" Riprap and 9" Bedding

RIPRAP GRADATIONS

12" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
15 INCH	100
12 INCH	70 – 100
8 INCH	45 – 75
6 INCH	30 – 55
3 INCH	10 – 30
1-1/2 INCH	0 – 10

RIPRAP GRADATIONS

8" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
10 INCH	100
8 INCH	70 – 100
6 INCH	50 – 75
3 INCH	20 – 40
1-1/2 INCH	0 – 15

BEDDING GRADATIONS

6" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
3 INCH	100
1-1/2 INCH	55 – 100
3/4 INCH	25 – 60
3/8 INCH	5 – 30
No.4	0 – 10

BEDDING GRADATIONS

9" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
6 INCH	100
3 INCH	65 – 100
1-1/2 INCH	40 – 60
3/4 INCH	25 – 40
No.4	0 – 12

Figure 4.4-17 Riprap Gradation Tables for 6", 8", 9", and 12" Thickness of Riprap

RIPRAP GRADATIONS

30" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
36 INCH	100
30 INCH	65 – 100
24 INCH	45 – 75
18 INCH	25 – 50
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS

18" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
21 INCH	100
18 INCH	65 – 100
12 INCH	35 – 65
8 INCH	15 – 40
6 INCH	5 – 25
4 INCH	0 – 15

RIPRAP GRADATIONS

36" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
44 INCH	100
36 INCH	65 – 100
30 INCH	50 – 80
18 INCH	25 – 45
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS

24" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65 – 100
18 INCH	45 – 75
12 INCH	25 – 50
8 INCH	10 – 30
6 INCH	0 – 15

Figure 4.4-18 Riprap Gradation Tables for 18", 24", 30", and 36" Thickness of Riprap

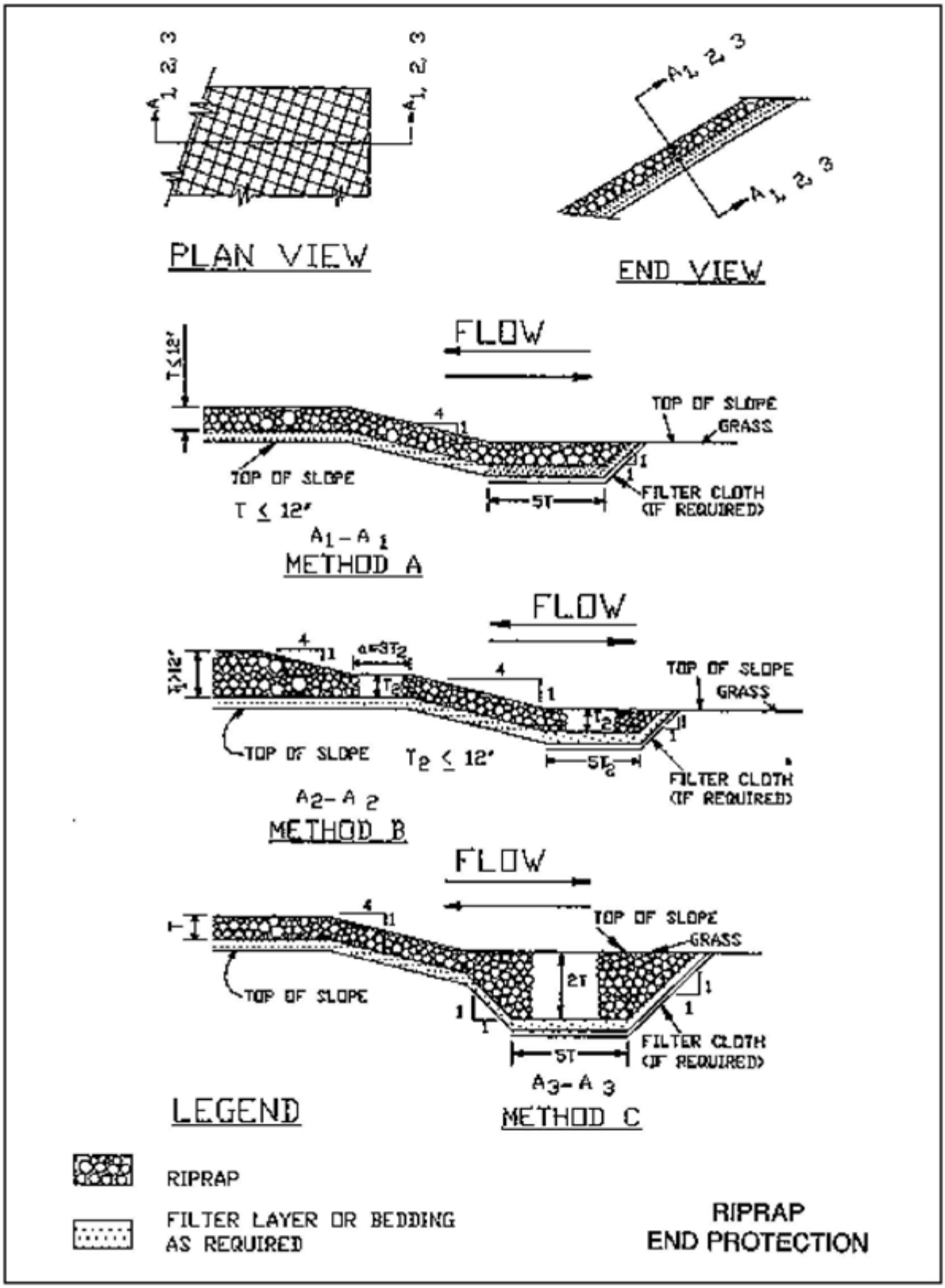


Figure 4.4-19 Typical Riprap Design Cross Sections

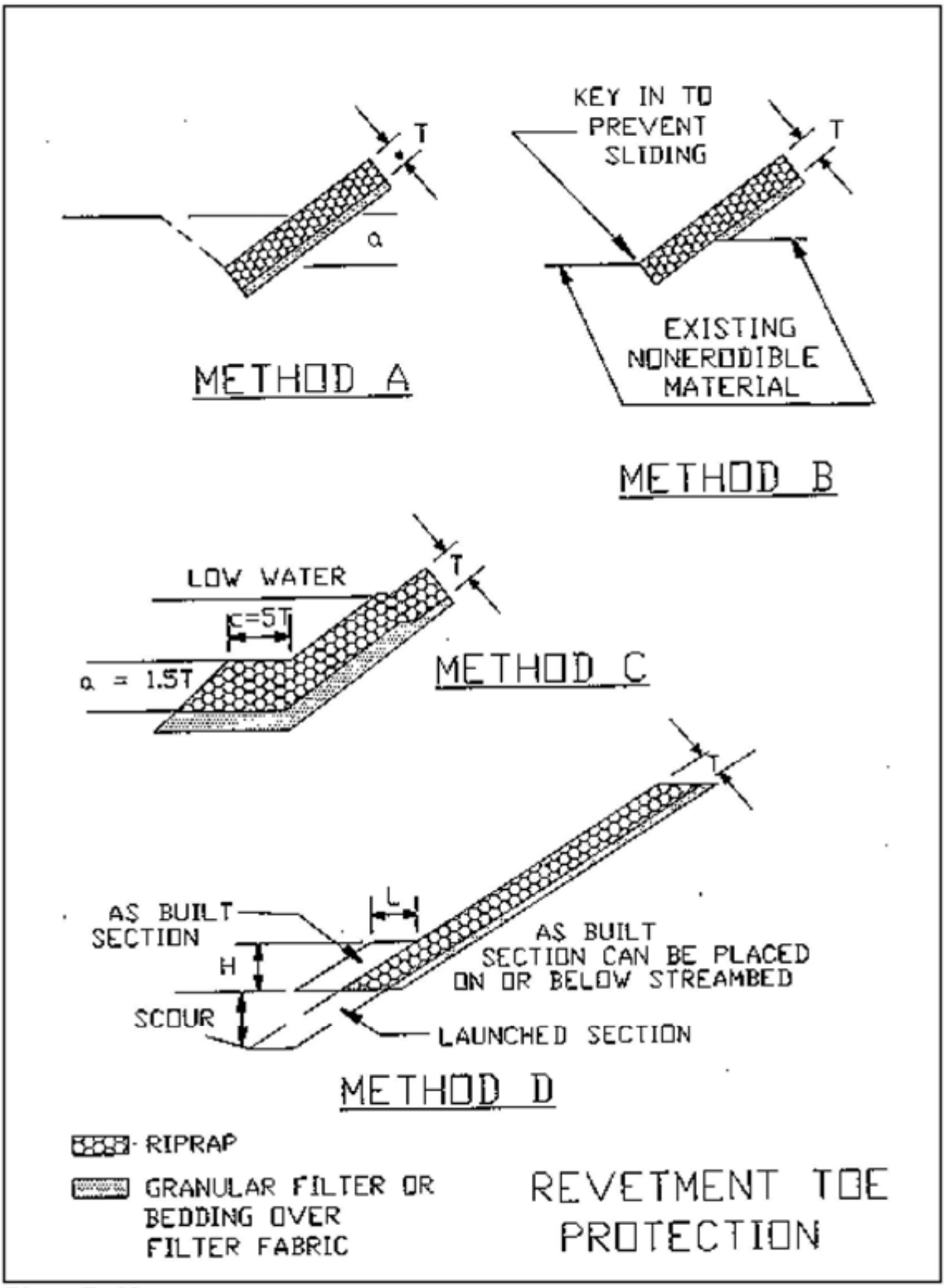


Figure 4.4-20 Typical Riprap Design for Revetment Toe Protection

4.4.9 Gabion Design

4.4.9.1 Introduction

Gabions come in three basic forms, the gabion basket, gabion mattress, and sack gabion. All three types consist of wire mesh baskets filled with cobble or small boulder material. The fill normally consists of rock material but other materials such as bricks have been used to fill the baskets. The baskets are used to maintain stability and to protect streambanks and beds.

The difference between a gabion basket and a gabion mattress is the thickness and the aerial extent of the basket. A sack gabion is, as the name implies, a mesh sack that is filled with rock material. The benefit of gabions is that they can be filled with rocks that would individually be too small to withstand the erosive forces of the stream. The gabion mattress is shallower (0.5 to 1.5 ft) than the basket and is designed to protect the bed or banks of a stream against erosion.

Gabion baskets are normally much thicker (about 1.5 to 3 ft) and cover a much smaller area. They are used to protect banks where mattresses are not adequate or are used to stabilize slopes (Figure 4.4-21), construct drop structures, pipe outlet structures, or nearly any other application where soil must be protected from the erosive forces of water. References to gabions in this manual refer generally to both mattresses and baskets. Sack gabions are rarely used in the United States and are not discussed.

Gabion baskets can be made from either welded or woven wire mesh. The wire is normally galvanized to reduce corrosion but may be coated with plastic or other material to prevent corrosion and/or damage to the wire mesh containing the rock fill. New materials such as Tensar, a heavy-duty polymer plastic material, have been used in some applications in place of the wire mesh. If the wire baskets break, either through corrosion, vandalism, or damage from debris or bed load, the rock fill in the basket can be lost and the protective value of the method endangered. Gabions are often used where available rock size is too small to withstand the erosive and tractive forces present at a project site. The available stone size may be too small due to the cost of transporting larger stone from remote sites, or the desire to have

Figure 4.4-21 Gabion Baskets Installed for Slope Stabilization along a Stream

a project with a smoother appearance than obtained from riprap or other methods. Gabions also require about one third the thickness of material when compared to riprap designs. Riprap is often preferred, however, due to the low labor requirements for its placement.

The science behind gabions is fairly well established, with numerous manufacturers providing design methodology and guidance for their gabion products. Dr. Stephen T. Maynord of the U.S. Army Corps of Engineers Research and Development Center in Vicksburg, Mississippi, has also conducted research to develop design guidance for the installation of gabions. Two general methods are typically used to determine the stability of gabion baskets in stream channels, the critical shear stress calculation and the critical velocity calculation. A software package known as CHANLPRO has been developed by Dr. Maynord (Maynord et al. 1998).

Manufacturers have generated extensive debate regarding the use and durability of welded wire baskets versus woven wire baskets in project design and construction. Project results seem to indicate that performance is satisfactory for both types of mesh.

The rocks contained within the gabions provide substrates for a wide variety of aquatic organisms. Organisms that have adapted to living on and within the rocks have an excellent home, but vegetation

may be difficult to establish unless the voids in the rocks contained within the baskets are filled with soil or a planting bed mixture.

If large woody vegetation is allowed to grow in the gabions, there is a risk that the baskets will break when the large woody vegetation is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The possibility of damage must be weighed against the desirability of vegetation on the area protected by gabions and the stability of the large woody vegetation. If large woody vegetation is kept out of the baskets, grasses and other desirable vegetation types may be established and provide a more aesthetic and ecologically desirable project than gabions alone.

4.4.9.2 Design

Primary design considerations for gabions and mattresses are: 1) foundation stability; 2) sustained velocity and shear-stress thresholds that the gabions must withstand; and 3) toe and flank protection. The base layer of gabions should be placed below the expected maximum scour depth. Alternatively, the toe can be protected with mattresses that will fall into any scoured areas without compromising the stability of the bank or bed protection portion of the project. If bank protection does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

The use of a filter fabric behind or under the gabion baskets to prevent the movement of soil material through the gabion baskets is an extremely important part of the design process. This migration of soil through the baskets can cause undermining of the supporting soil structure and failure of the gabion baskets and mattresses.

<u>Primary Design Considerations</u>

The major consideration in the design of gabion structures is the expected velocity at the gabion face. The gabion must be designed to withstand the force of the water in the stream.

Since gabion mattresses are much shallower and more subject to movement than gabion baskets, care should be taken to design the mattresses such that they can withstand the forces applied to them by the water. However, mattresses have been used in application where very high velocities are present and have performed well. But, projects using gabion mattresses should be carefully designed.

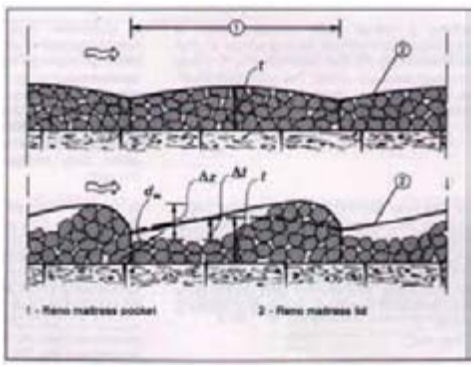
The median stone size for a gabion mattress can be determined from the following equation:

$$d_m = S_f C_s C_v d[(\Upsilon_w/(\Upsilon_s - \Upsilon_w))^{0.5}(V/\sqrt{(gdK_1)})]^{2.5} \qquad (4.4.23)$$

where:

- d_m = average rock diameter in gabions (ft)
- S_f = safety factor (1.1 minimum)
- C_s = stability coefficient (usually 0.1)
- C_v = velocity distribution coefficient = 1.283-0.2log(r/w) (minimum of 1.0) and equals 1.25 at end of dikes and concrete channels
- r = center-line bend radius of main channel flow (ft)
- w = water surface width of main channel (ft)
- d = local flow depth at V (ft)
- g = acceleration due to gravity (32.2 ft/s^2)
- V = depth-averaged velocity (ft/s)
- K_1 = side slope correction factor (Table 4.4-8)
- Υ_w = unit weight of water (62.4 lb/ft^3)
- Υ_s = unit weight of stone (lb/ft^3)

Table 4.4-8 Values of K_1 for various Side Slopes to be used in Equation 4.4-23	
Side Slope	**K_1**
1V : 1H	0.46
1V : 1.5H	0.71
1V : 2H	0.88
1V : 3H	0.98
<1V : 4H	1.0



Equation 4.4-23 was developed to design stone size such that the movement of filler stone in the mattresses is prevented. This eliminates deformation that can occur when stone sizes are not large enough to withstand the forces of the water. The result of mattresses deformation is stress on the basket wire and increases in resistance to flow and the likelihood of basket failure. The upper portion of Figure 4.4-22 shows an undeformed gabion, while the lower portion shows how gabions deform under high-velocity conditions. Maccaferri Gabions gives guidance on sizing stone and allowable velocities for gabion baskets and mattresses, shown in Table 4.4-9.

Figure 4.4-22 Gabion Mattress Showing Deformation of Mattress Pockets under High Velocities

Table 4.4-9 Stone Sizes and Allowable Velocities for Gabions					
Type	**Thickness (ft)**	**Filling Stone Range**	**D_{50}**	**Critical Velocity**	**Limit Velocity**
Mattress	0.5	3 – 4”	3.4”	11.5	13.8
	0.5	3 – 6”	4.3”	13.8	14.8
	0.75	3 – 4”	3.4”	14.8	16
	0.75	3 – 6”	4.7”	14.8	20
	1.0	3 – 5”	4”	13.6	18
	1.0	4 – 6”	5”	16.4	21
Basket	1.5	4 – 8”	6”	19	24.9
	1.5	5 – 10”	7.5”	21	26.2

When the data in Table 4.4-9 are compared to Equation 4.4-22, if V = 11.5, C_s = 0.1, C_v = 1.0, K_1 = 0.71, ,Υ_s = 150 lb/ft^3 and S_f = 1.1, the local flow depth must be on the order of 25 ft in order to arrive at the stone diameter of 3.4 in. shown in Table 4.4-9. Designers should use Equation 4.4-23 to take the depth of flow into account. Table 4.4-9 does, however, give some general guidelines for fill sizes and is a quick reference for maximum allowable velocities.

Maccaferri also gives guidance on the stability of gabions in terms of shear stress limits. The following equation gives the shear for the bed of the channel as:

$$\tau_b = \Upsilon_w S d \qquad (4.4.24)$$

where S = bed or water surface slope through the reach (ft/ft)

The bank shear is generally taken as 75 percent of the bed shear, i.e.,

$$\tau_m = 0.75\ \tau_m \qquad (4.4.25)$$

These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\Upsilon_s - \Upsilon_w) d_m \qquad (4.4.26)$$

with critical shear stress for the banks given as:

$$\tau_s = \tau_c \sqrt{(1-(\sin^2\Theta/0.4304))} \qquad (4.4.27)$$

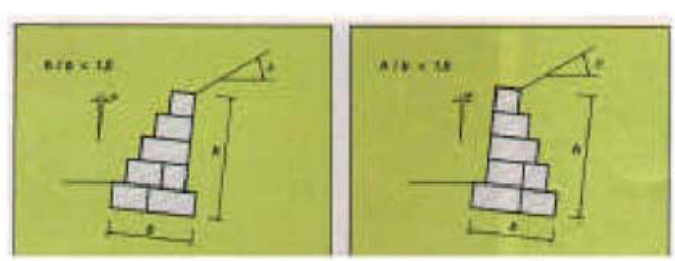
where Θ = angle of the bank rotated up from horizontal.

A design is acceptable if $\tau_b < \tau_c$ and $\tau_m < \tau_s$. If either $\tau_b > \tau_c$ or $\tau_m > \tau_s$ then a check must be made to see if they are less than 120 percent of τ_b and τ_s. If the values are less than 120 percent of τ_b and τ_s the gabions will not be subject to more than what Maccaferri defines as "acceptable" deformation. However it is recommended that the stone size be increased to limit deformation if possible.

Research has indicated that stone in the gabion mattress should be sized such that the largest stone diameter is not more than about two times the diameter of the smallest stone diameter and the mattress should be at least twice the depth of the largest stone size. The size range should, however, vary by about a factor of two to ensure proper packing of the stone material into the gabions. Since the mattresses normally come in discrete sizes, i.e. 0.5, 1.0, and 1.5 ft in depth, normal practice is to size the stone and then select the basket depth that is deep enough to be at least two times the largest stone diameter. The smallest stone should also be sized such that it cannot pass through the wire mesh.

Stability of Underlying Bed and Bank Material

Another critical consideration is the stability of the gabion foundation. This includes both geotechnical stability and the resistance of the soil under the gabions to the erosive forces of the water moving through the gabions. If there is any question regarding the stability of the foundation, i.e. possibility of rotational failures, slip failures, etc., a qualified geotechnical engineer should be consulted prior to and during the design of the bank/channel protection. Several manufacturers give guidance on how to check for geotechnical failure.



One of the critical factors in determining stability is the velocity of the water that passes through the gabions and reaches the soil behind the gabion. The water velocity under the filter fabric, i.e. water that moves through the gabions and filter fabric, is estimated to be one-fourth to one-half of the velocity at the mattress/filter interface.

Figure 4.4-23 Front-step and Rear-step Gabion Layout

The velocity at the mattress/filter interface, V_b, is estimated to be

$$V_b = 1.486\sqrt{S}(d_m/2)^{2/3}/n_f \qquad (4.4.28)$$

where n_f = 0.02 for filter fabric, 0.022 for gravel filter material

If the underlying soil material is not stable, additional filter material must be installed under the gabions to ensure soil stability.

The limit for velocity on the soil is different for each type of soil. The limit for cohesive soils is obtained from a chart, while maximum allowable velocities for other soil types are obtained by calculating V_e, the maximum velocity allowable at the soil interface, and comparing it to V" the residual velocity on the bed, i.e. under the gabion mattress and under the filter fabric. V_e for loose soils is equal to $16.1d^2$ while V_f is calculated by:

$$V_f = 1.486S\sqrt{V_a}(d_m/2)^{2/3}/n_f \qquad (4.4.29)$$

where V_a = average channel velocity (ft/s)

If V_f is larger than two to four times V_e, a gravel filter is required to further reduce the water velocity at the soil interface under the gabions until V_f is in an acceptable range. To check for the acceptability of the filter use the average gravel size for d_m in equation 4.4-28. If the velocity V_f is still too high, the gravel size should be reduced to obtain an acceptable value for V_f.

Other Design Considerations

It may be possible to combine gabions with less harsh methods of bank protection on the upper bank and still achieve the desired result of a stable channel. Provisions for large woody vegetation and a more aesthetically pleasing project may also be used on the upper banks or within the gabions. However, the stability of vegetation or other upper bank protection should be carefully analyzed to ensure stability of the upper bank area. A failure in the upper bank region can adversely affect gabion stability and lead to project failure.

4.4.10 Uniform Flow – Example Problems

Example 1 -- Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ft. Solve using Figure 4.4-2:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.
2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/s from the velocity scale.

Example 2 -- Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 ft/ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

1. From Table 4.4-3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.
2. Assume an n value of 0.035 and find the value of vR from Figure 4.4-1, vR = 5.4
3. Use equation 4.4.9 to calculate the value of R: R = 5.4/4 = 1.35 ft
4. Use equation 4.4.10 to calculate the value of vR:
   $vR = [1.49\ (1.35)^{5/3}\ (0.015)^{1/2}]/0.035 = 8.60$
5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n Value	vR (Figure 4.4-1)	R (eq. 4.4.9)	vR (eq. 4.4.10)
0.035	5.40	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

6. Use Figure 4.4-3 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   Qn = (50) (0.040) = 2.0, S = 0.015
   For b = 10 ft, d = (10) (0.098) = 0.98 ft, b = 8 ft, d = (8) (0.14) = 1.12 ft

   Select:

   b = 10 ft, such that R is approximately 0.80 ft
   z = 3
   d = 1 ft
   v = 3.9 ft/s (equation 4.4.1)
   Fr = 0.76 (equation 4.4.8)
   Flow is subcritical

   Design capacity calculations for this channel are presented in Example 3 below.

Example 3 -- Grassed Channel Design Capacity

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.

Assume a depth of 1.0 ft and calculate the following (see Figure 4.4-5):

A = (b + zd) d = [10 + (3) (1)] (1) = 13.0 square ft

R = $[(b + zd)\ d]/\{b + [2d(1 + z^2)^{0.5}]\} = \{[10+(3)(1)]1\}/\{10+[(2)(1)(1+3^2)^{0.5}]\}$

R = 0.796 ft

Find the velocity: v = Q/A = 50/13.0 = 3.85 ft/s

Find the value of vR: vR = (3.85) (0.796) = 3.06

Using the vR product from Step 3, find Manning's n from Figure 4.4-1 for retardance Class C (n = 0.047)

Use Figure 4.4-2 or equation 4.4.1 to find the velocity for S = 0.015, R = 0.796, and n = 0.047: <u>v = 3.34 ft/s</u>

Since 3.34 ft/s is less than 3.85 ft/s, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 4.4-1)	Velocity (Eq. 4.4.1)
1.0	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.047	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 ft. Design data for the trapezoidal channel are summarized as follows:

   Vegetation lining = grass mixture, v_m = 4 ft/s

   Q = 50 cfs

   b = 10 ft, d = 1.3 ft, z = 3, S = 0.015
   Top width = (10) + (2) (3) (1.3) = 17.8 ft

   n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (equation 4.4.8)
   n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (equation 4.4.8)

Example 4 -- Riprap Design

A natural channel has an average bankfull channel velocity of 8 ft per second with a top width of 20 ft and a bend radius of 50 ft. The depth over the toe of the outer bank is 5 ft. Available stone weight is 170 lbs/ft^3. Stone placement is on a side slope of 2:1 (horizontal:vertical). Determine riprap size at the outside of the bend.

1. Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.
2. Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 4.4-7, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.
3. Determine the correction coefficient for the specific weight of 170 lbs from Figure 4.4-8 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.
4. Determine minimum d_{30} from Figure 4.4-9 or equation 4.4.13 as about 10 inches.
5. Use a gradation with a minimum d_{30} size of 10 inches.
6. *(Optional)* Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 4.4-10, this gradation would be acceptable if the blanket thickness was increased from the original d_{100} (diameter of the largest stone) thickness by 35% (a ratio of 1.35 on the horizontal axis).
7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 4.4-6.

4.4.11 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-RAS, developed by the U.S. Army Corps of Engineers and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step method (Chow, 1959, TxDOT, 2002). In the Direct Step method an increment of water depth is chosen, and the distance over which the depth change occurs is computed. This method is often used in association with culvert hydraulics. It is most accurate when the slope and depth increments are small. It is appropriate for prismatic channel sections which occur in most conduits, and can be useful when

estimating both supercritical and subcritical profiles. For supercritical flow, the water surface profile is computed downstream. For subcritical flow, the water surface profile is computed upstream.

For an irregular nonuniform channel, the Standard Step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for Standard Step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity the designed waterway. Channel cross sections will be required at each location along the waterway where there are changes in channel shape or dimension, changes in the flowline slope, and changes in vegetation or channel lining. These sections are in addition to any sections necessary to define obstructions such as culverts, bridges, damns, energy dissipation features, or aerial crossings (pipelines). Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

4.4.12 Rectangular, Triangular and Trapezoidal Open Channel Design

4.4.12.1 Introduction

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial and error solution of Manning's Equation must be used. However, it is anticipated that available software programs will be the first choice for solving these design computations.

4.4.12.2 Description of Figures

Figures given in FHWA, HDS No. 3, 1973 and Atlanta Regional Commission, 2001 are for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft). A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but also between the inclined lines representing depth and slope.

The chart for a triangular cross section (see Figure 3.2-1) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V-shaped sections by following the instructions on the chart.

4.4.12.3 Instructions for Rectangular and Trapezoidal Figures

Figures such as Figure 4.4-24 provide a solution of the Manning equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the figure for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the figures can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity V from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

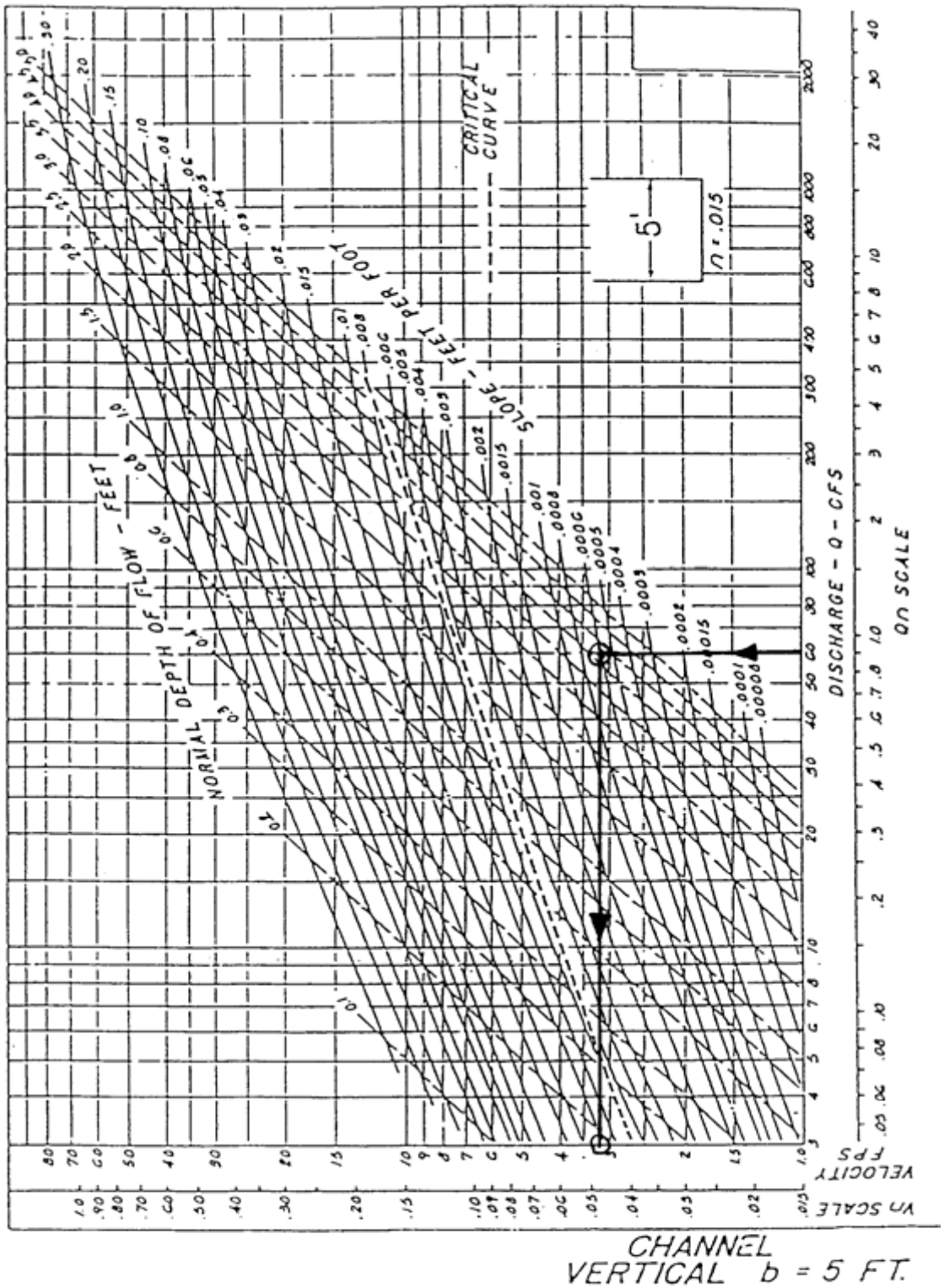
Example Design Problem 1

Given: A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure:

1. From subsection 4.4.12, select the rectangular figure for a 5-ft width (Figure 4.4-24).
2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read d_n = 3.7 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.
4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.



Source: Federal Highway Administration

Figure 4.4-24 Example Nomograph #1

Example Design Problem 2

Given: A trapezoidal channel with 2:1 side slopes and a 4 ft bottom width, with n = 0.030, 0.2% slope (S = 0.002), discharging 50 cfs.

Find: Depth, velocity, type flow.

Procedure:

1. Select the trapezoidal figure for b = 4 ft (see Figure 4.4-25).
2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d_n = 2.2 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/s, on the ordinate scale. The intersection lies below the critical curve, and the flow is therefore subcritical.

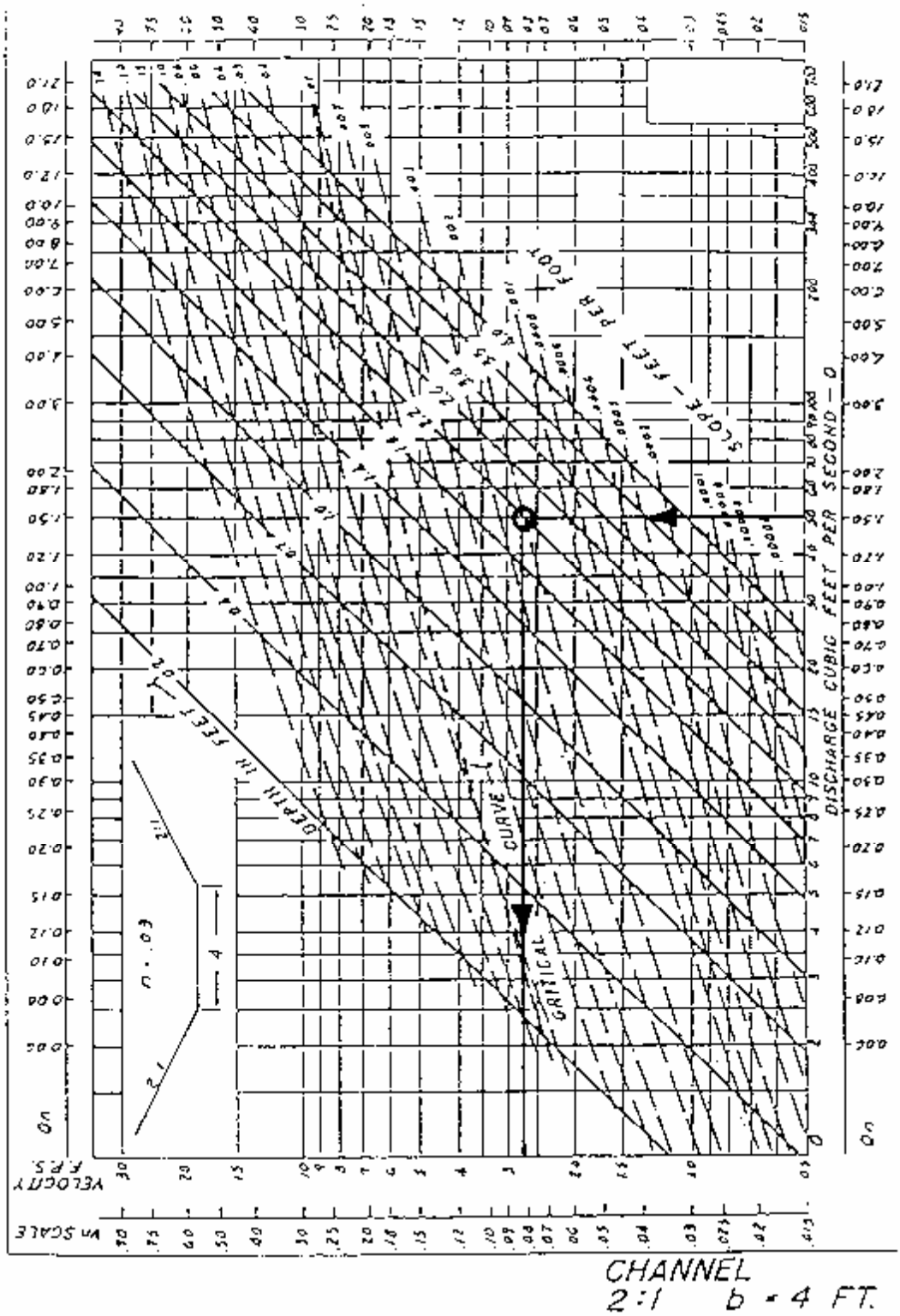
Example Design Problem 3

Given: A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, 0.5% slope (S = 0.005), discharging 80 cfs.

Find: Depth velocity and type of flow

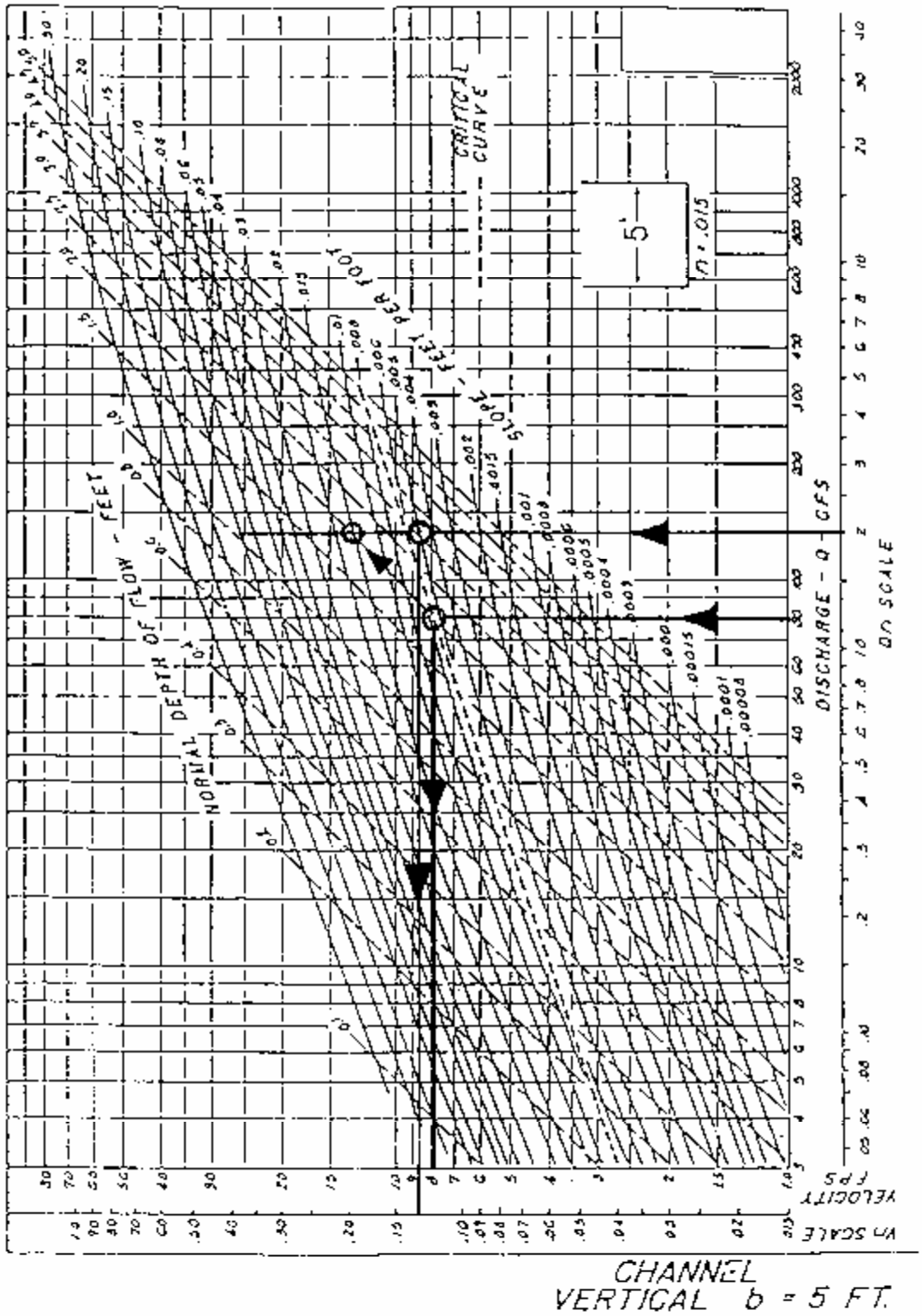
Procedure:

1. Select the rectangular figure for a 5 ft width (Figure 4.4-26).
2. Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.
3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read d_n = 3.1 ft.
4. Move horizontally from the intersection and read Vn = .13, then Vn/n = 0.13/0.025 = 5.2 ft/s.
5. Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 4.4-13, d_c = 2.0 ft and V_c = 7.9 ft/s. The normal velocity, 5.2 ft/s (from step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.
6. To determine the critical slope for Q = 80 cfs and n = 0.025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding d_c (2.0 ft) at this point. Follow along this d_c line to its intersection with a vertical line through Qn = 2.0 (step 2), at this intersection read the slope value S_c = 0.015.



Source: Federal Highway Administration

Figure 4.4-25 Example Nomograph #2



Source: Federal Highway Administration

Figure 4.4-26 Example Nomograph #3

4.4.12.4 Grassed Channel Figures

The Manning equation can be used to determine the capacity of a grass-lined channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass-lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass-lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

To aid in the design of grassed channels, the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in subsection 4.4.14. Following is a brief description of general design criteria, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial-and-error solution of the Manning equation must be used.

4.4.12.5 Description of Figures

A set of figures in FHWA, NDS No. 3, 1973 and Atlanta Regional Commission, 2001 are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (n) changes as higher velocities and/or greater depths change the condition of the grass. The effect of velocity and depth of flow on n is evaluated by the product of velocity and hydraulic radius V times R. The variation of Manning's n with the retardance (Table 4.4-6) and the product V times R is shown in Figure 4.4-1. As indicated in Table 4.4-6, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. Table 4.4-6 is used to determine retardance classification.

The grassed channel figures each have two graphs, the upper graph for retardance Class D and the lower graph for retardance Class C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic.

Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

4.4.12.6 Instructions for Grassed Channel Figures

The grassed channel figures like those in Figure 4.4-11 provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have length sufficient to establish uniform flow. The figures are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations: (1) selecting a section that has the capacity to carry the design discharge on the available slope and (2) checking the velocity in the channel to ensure that the grass lining will not be eroded. Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance Class C and

the velocity using retardance Class D. The calculated velocity should then be checked against the permissible velocities listed in Tables 4.4-2 and 4.4-3. The use of the figures is explained in the following steps:

Step 1 Select the channel cross section to be used and find the appropriate figure.

Step 2 Enter the lower graph (for retardance Class C) on the figure with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is subcritical; if it is above, the flow is supercritical.

Step 3 To check the velocity developed against the permissible velocities (Tables 4.4-2 and 4.4-3), enter the upper graph on the same figure and repeat Step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

Example Design Problem 1

Given: A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4:1 side slopes, and a 4 ft bottom width on slope of 0.02 ft per foot (S=.02), discharging 20 cfs.

Find: Depth, velocity, type of flow, and adequacy of grass to prevent erosion

Procedure:

1. From subsection 4.4.13 select figure for 4:1 side slopes (see Figure 4.4-27).
2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S=0.02. At this intersection read d_n = 1.0 ft, and normal velocity V_n 2.6 ft/s.
3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance Class D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/s. This is about three-quarters of that listed as permissible, 4.0 ft/s in Table 4.4-3.

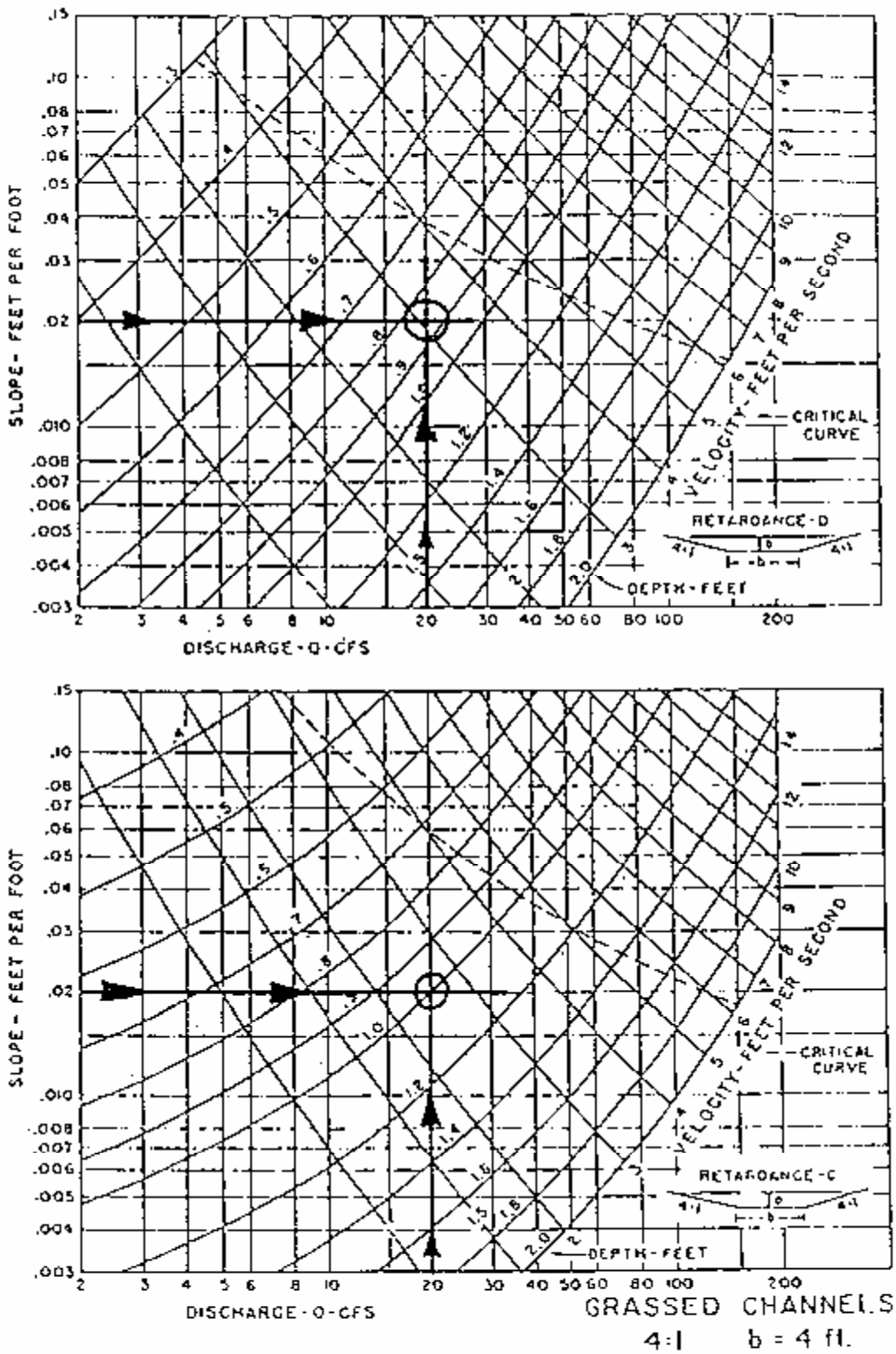
Example Design Problem 2

Given: The channel and discharge of Example 1.

Find: The maximum grade on which the 20 cfs could safely be carried

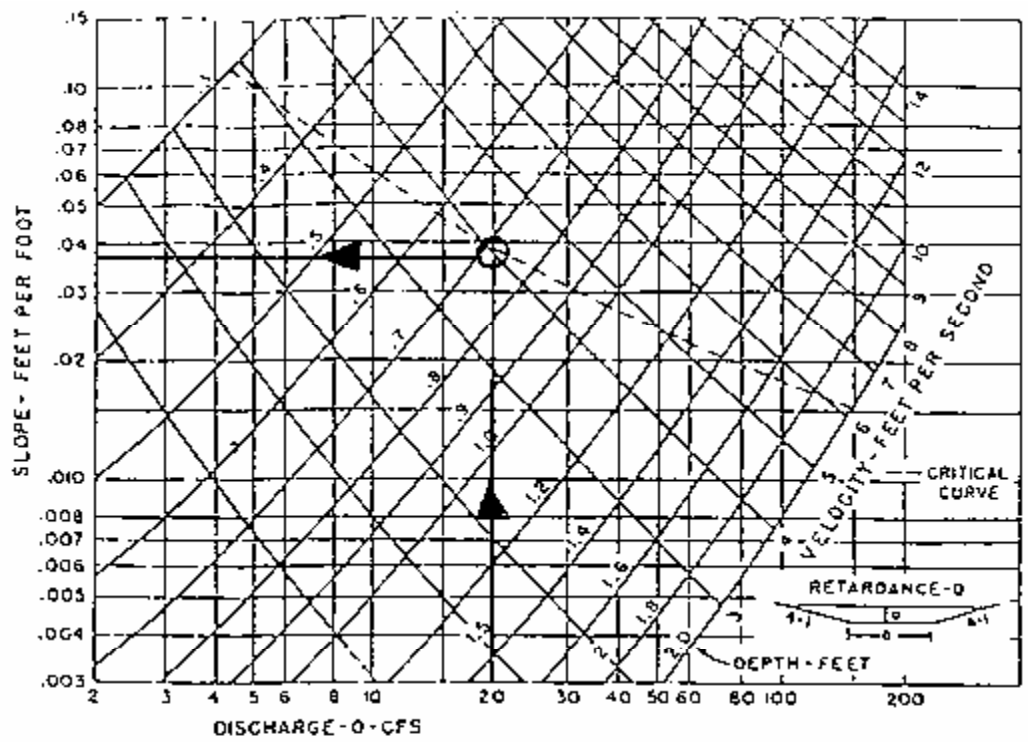
Procedure:

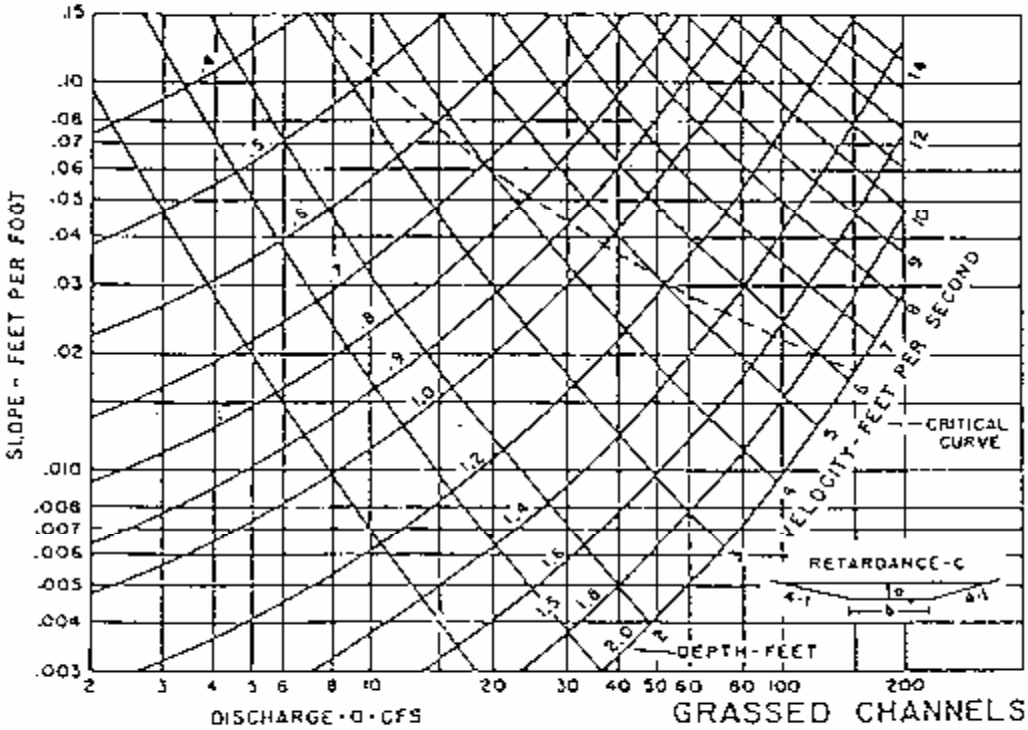
With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 4.4-3). On the upper graph of Figure 4.4-28 for short grass, the intersection of the 20 cfs line and the 4 ft/s line indicates a slope of 3.7% and a depth of 0.73 ft.



Source: Federal Highway Administration

Figure 4.4-27 Example Nomograph #4





GRASSED CHANNELS
4:1 b = 4 ft.

Source: Federal Highway Administration

Figure 4.4-28 Example Nomograph #5

References

Atlanta Regional Commission, 2001, Georgia Stormwater Management Manual, Volume 2, Technical Handbook, First Edition, pages 4.4-40 through 4.4-73 (http://www.georgiastormwater.com/vol2/4-4.pdf).

Chow, V. T., 1959. Open Channel Hydraulics. McGraw-Hill Book Co. New York, New York.

French, R. H., 1985. Open Channel Hydraulics. McGraw-Hill Book Co. New York, New York.

Federal Highway Administration, 1989. Bridge Waterways Analysis Model (WSPRO), Users Manual, FHWA IP-89-027.

Gregory, G.T., June 1987. Stone Riprap Design Guidelines. Unpublished paper, Fort Worth, Texas.

Harza Engineering Company, 1972. Storm Drainage Design Manual. Prepared for the Erie and Niagara Counties Regional Planning Bd. Harza Engineering Company, Grand Island, NY.

Maynord, S. T., 1987. Stable Riprap Size for Open Channel Flows. Ph.D. Dissertation. Colorado State University, Fort Collins, CO.

Mays, L. W., ed, 1999. Hydraulic Design Handbook. McGraw-Hill, New York, New York.

Morris, J. R., 1984. A Method of Estimating Floodway Setback Limits in Areas of Approximate Study. In Proceedings of 1984 International Symposium on Urban Hydrology, Hydraulics and Sediment Control. Lexington, Kentucky: University of Kentucky.

Peterska, A. J., 1978. Hydraulic Design of Stilling Basins and Energy Dissipators. Engineering Monograph No. 25. U. S. Department of Interior, Bureau of Reclamation. Washington, DC.

Reese, A. J., 1984. Riprap Sizing, Four Methods. In Proceedings of ASCE Conference on Water for Resource Development, Hydraulics Division, ASCE. David L. Schreiber, ed.

Reese, A. J., 1988. Nomographic Riprap Design. Miscellaneous Paper HL 88-2. Vicksburg, Mississippi: U. S. Army Engineers, Waterways Experiment Station.

Texas Department of Transportation, 2002, Hydraulic Design Manual, Revision 2002-2, Austin, Texas

U. S. Army Corps of Engineers, 1991/1994, Hydraulic Design of Flood Control Channels, Engineer Manual, EM 1110-2-1601 (htt;://www.usace.army.mil/usace-docs/).

U. S. Army Corps of Engineers Research and Development Center, 2000. Gabions for Streambank Erosion Control, ERDC TN-EMRRP-SR-22, Vicksburg, Mississippi.

U. S. Department of Transportation, Federal Highway Administration, 1961. Design Charts For Open Channel Flow. Hydraulic Design Series No. 3. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1984. Guide for Selecting Manning's Roughness Coefficients For Natural Channels and Flood Plains. FHWA-TS-84-204. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1988. Design of Stable Channels with Flexible Linings. Hydraulic Engineering Circular No. 15. Washington, DC.

Wright-McLaughlin Engineers, 1969. Urban Storm Drainage Criteria Manual, Vol. 2. Prepared for the Denver Regional Council of Governments. Wright-McLaughlin Engineers, Denver, CO

Section 4.5
Storage Design

4.5.1 General Storage Concepts

4.5.1.1 Introduction

This section provides general guidance on storm water runoff storage for meeting storm water management control objectives (i.e., water quality protection, downstream streambank protection, and flood control).

Storage of storm water runoff within a storm water management system is essential to providing the extended detention of flows for water quality protection and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural storm water controls and/or nonstructural features and landscaped areas. Figure 4.5-1 illustrates various storage facilities that can be considered for a development site.

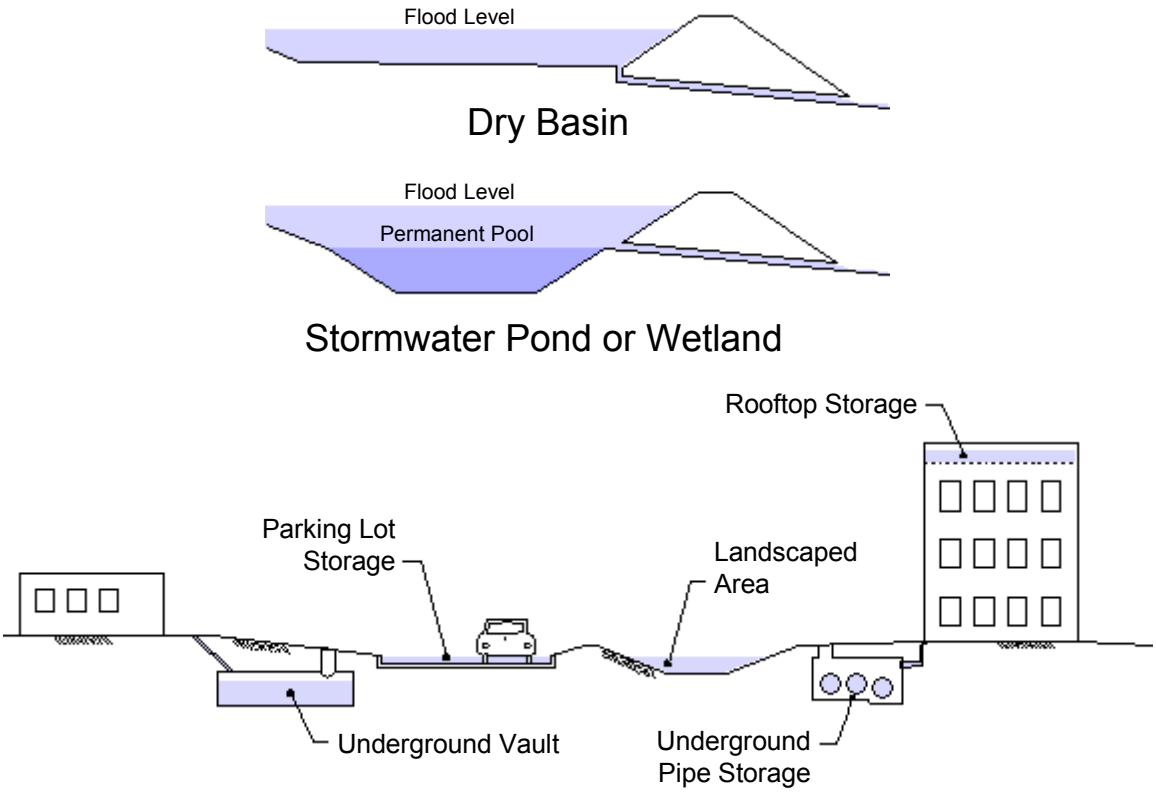


Figure 4.5-1 Examples of Typical Storm Water Storage Facilities

4.5.1.2 Storage Classification

Storm water storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

Storm water *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet streambank protection criteria, and flood criteria where required.

Extended detention (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet streambank protection criteria. Some structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) also include extended detention storage of a portion of the water quality protection volume.

Retention facilities are designed to contain a permanent pool of water, such as storm water ponds and wetlands, which is used for water quality protection.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage storm water runoff from multiple projects and/or properties. A discussion of regional storm water controls is found in Section 5.1.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 4.5-2 illustrates on-line versus off-line storage.

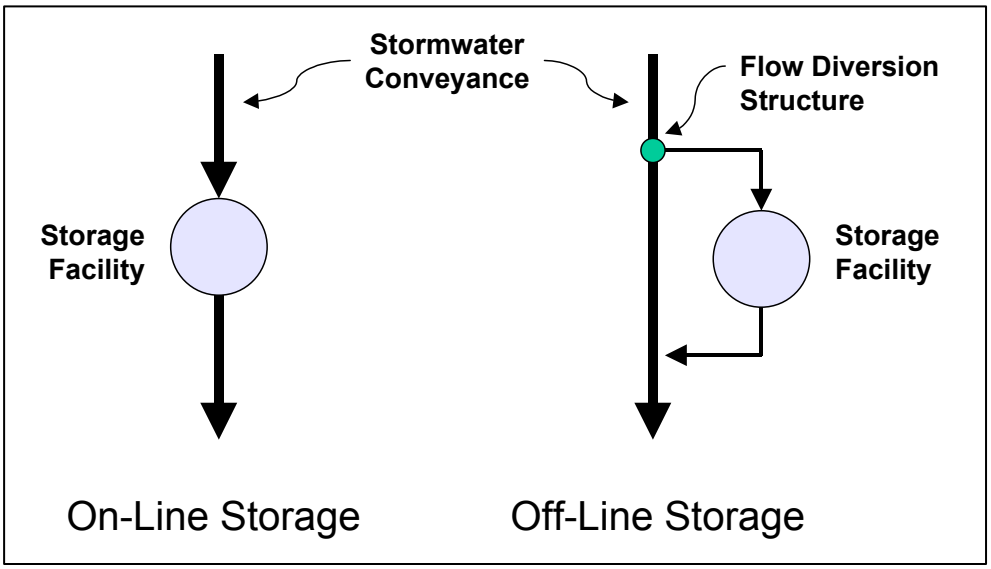


Figure 4.5-2 On-Line versus Off-Line Storage

4.5.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 4.5-3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

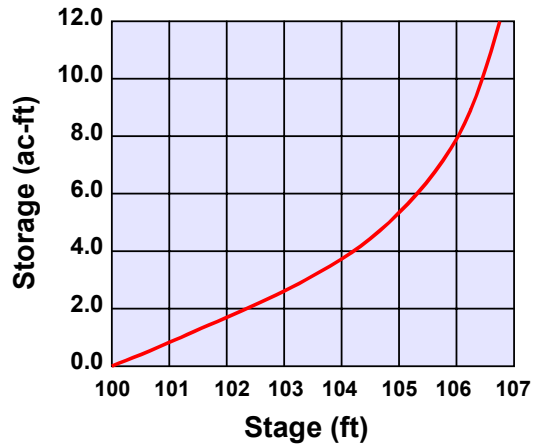


Figure 4.5-3 Stage-Storage Curve

The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see Figure 4.5-4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (4.5.1)$$

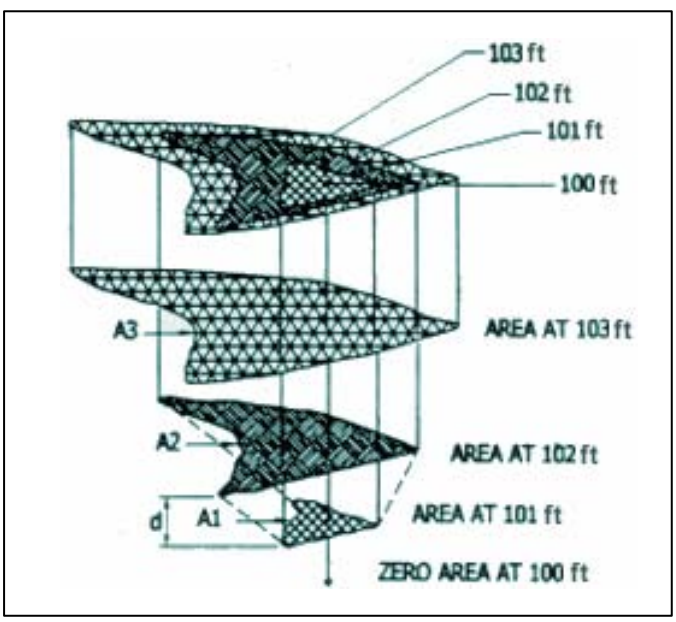


Figure 4.5-4 Double-End Area Method

where:

$V_{1,2}$ = storage volume (ft^3) between elevations 1 and 2
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)
d = change in elevation between points 1 and 2 (ft)

The frustum of a pyramid formula is expressed as:

$$V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3 \qquad (4.5.2)$$

where:

V = volume of frustum of a pyramid (ft^3)
d = change in elevation between points 1 and 2 (ft)
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3 \qquad (4.5.3)$$

where:

V = volume of trapezoidal basin (ft^3)
L = length of basin at base (ft)
W = width of basin at base (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$V = 1.047\ D\ (R_1^2 + R_2^2 + R_1R_2) \qquad (4.5.4)$$

$$V = 1.047\ D\ (3\ R_1^2 + 3ZDR_1 + Z_2D^2) \qquad (4.5.5)$$

where:

R_1, R_2 = bottom and surface radii of the conic section (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

4.5.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 4.5-5). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 4.6, *Outlet Structures*.

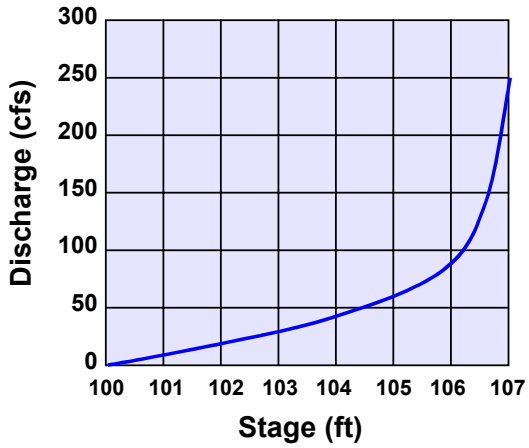


Figure 4.5-5 Stage-Discharge Curve

4.5.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.5-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.5-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	sec
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K	Coefficient	-
I	Inflow rate	cfs
L	Length	ft
Q, q	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
S, V_S	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_I	Duration of basin inflow	hrs
t_P	Time to peak	hrs
V_S, S	Storage volume	ft^3, in, acre-ft
V_r	Volume of runoff	ft^3, in, acre-ft
W	Width of basin	ft
Z	Side slope factor	-

4.5.3 General Storage Design Procedures

4.5.3.1 Introduction

This section discusses the general design procedures for designing storage to provide standard detention of storm water runoff for flood control (Q_f).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural storm water controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see subsection 2.1.9).

In multi-purpose multi-stage facilities such as storm water ponds, the design of storage must be integrated with the overall design for water quality protection objectives. See Chapter 5 for further guidance and criteria for the design of structural storm water controls.

4.5.3.2 Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

4.5.3.3 Design Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

Step 1 Compute inflow hydrograph for runoff from the "Conveyance" (e.g., Q_{p25}) and 100-year (Q_{p100}) design storms using the hydrologic methods outlined in Section 2.1. Both existing- and post-development hydrographs are required for both the "Conveyance" and 100-year design storms.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see subsection 4.5.4).

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

Step 4 Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the "Conveyance" design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.

Step 6 Perform routing calculations using the 100-year hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflows for the "Conveyance" and 100-year storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system to the location where the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system (see subsection 2.1.9).

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including North Central Texas.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

4.5.4 Preliminary Detention Calculations

4.5.4.1 Introduction

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

4.5.4.2 Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 4.5-6.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_i (Q_i - Q_O) \qquad (4.5.6)$$

where:

V_S = storage volume estimate (ft^3)

Q_i = peak inflow rate (cfs)

Q_O = peak outflow rate (cfs)

T_i = duration of basin inflow (s)

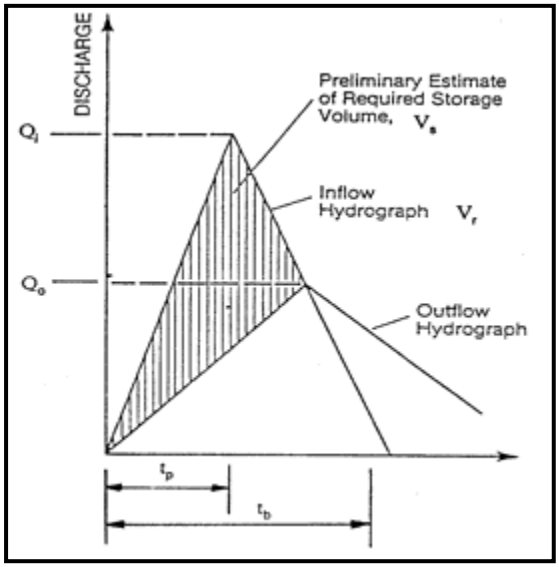


Figure 4.5-6 Triangular-Shaped Hydrographs (For Preliminary Estimate of Required Storage Volume)

4.5.4.3 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

Determine input data, including the allowable peak outflow rate, Q_O, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_p}{t_b}\right)^{0.753}} \qquad \textbf{(4.5.7)}$$

where:

V_S = volume of storage (in)

V_r = volume of runoff (in)

Q_O = outflow peak flow (cfs)

Q_i = inflow peak flow (cfs)

t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]

t_p = time to peak of the inflow hydrograph (hr)

Multiply the volume of runoff, V_r, times the ratio V_S/V_r, calculated in Step 2 to obtain the estimated storage volume V_S.

4.5.4.4 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_i, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph, t_p, and storage volume V_S.

Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff and Singh, 1976):

$$Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \qquad (4.5.8)$$

where:

- Q_O = outflow peak flow (cfs)
- Q_i = inflow peak flow (cfs)
- V_S = volume of storage (in)
- V_r = volume of runoff (in)
- t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
- t_p = time to peak of the inflow hydrograph (hr)

Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_O, for the selected storage volume.

References

Metropolitan Government of Nashville and Davidson County, 1988. Storm Water Management Manual - Volume 2 Procedures. Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill.

Wycoff, R. L. and U. P. Singh, 1976. Preliminary Hydrologic Design of Small Flood Detention Reservoirs. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49.

Section 4.6
Outlet Structures

4.6.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.6-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.6-1 Symbols and Definitions

Symbol	Definition	Units
A, a	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
B	Breadth of weir	ft
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K, k	Coefficient	-
L	Length	ft
n	Manning's n	-
Q, q	Peak inflow or outflow rate	cfs, in
V_u	Approach velocity	ft/s
WQ_v	Water quality protection volume	ac ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_g	Angle of the grate with respect to the horizontal	degrees

4.6.2 Primary Outlets

4.6.2.1 Introduction

Primary outlets provide the critical function of the regulation of flow for structural storm water controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the storm water facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for storm water storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

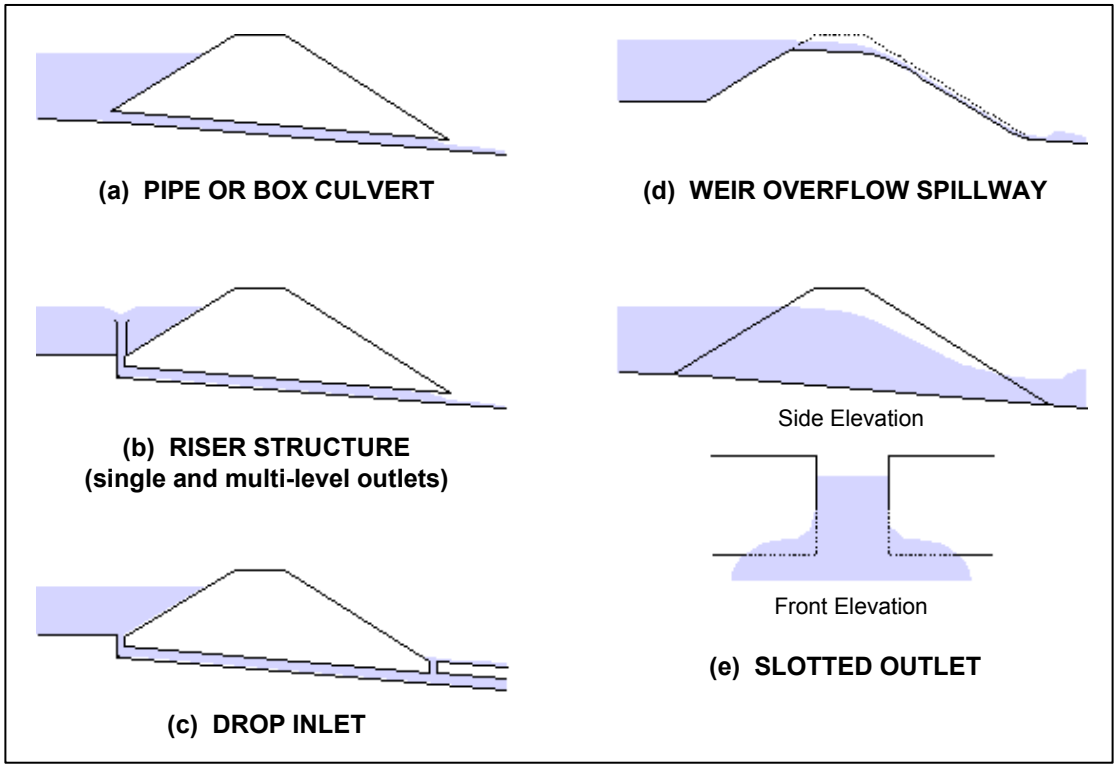


Figure 4.6-1 Typical Primary Outlets

4.6.2.2 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in storm water facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

The design professional must pay attention to material types and construction details when designing an outlet structure or device. Non-corrosive material and mounting hardware are key to device longevity, ease of operation, and low cost maintenance. Special attention must also be paid to not placing dissimilar metal materials together where a cathodic reaction will cause deterioration and destruction of metal parts.

Protective coatings, paints, and sealants must also be chosen carefully to prevent contamination of the storm water flowing through the structure/device. This is not only important while they are being applied, but also as these coating deteriorate and age over the functional life of the facility.

Each of these outlet types has a different design purpose and application:

- Water quality and streambank protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.

- Larger flows, such as flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

4.6.2.3 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 4.6-2(a), the orifice discharge can be determined using the standard orifice equation below.

$$Q = CA\,(2gH)^{0.5} \qquad (4.6.1)$$

where:

Q = the orifice flow discharge (cfs)
C = discharge coefficient
A = cross-sectional area of orifice or pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
D = diameter of orifice or pipe (ft)
H = effective head on the orifice, from the center of orifice to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 4.6-2(b).

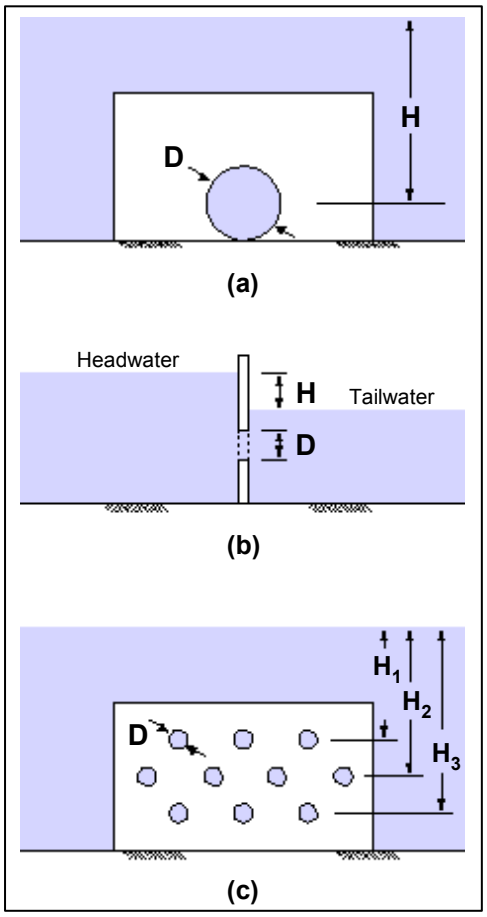


Figure 4.6-2 Orifice Definitions

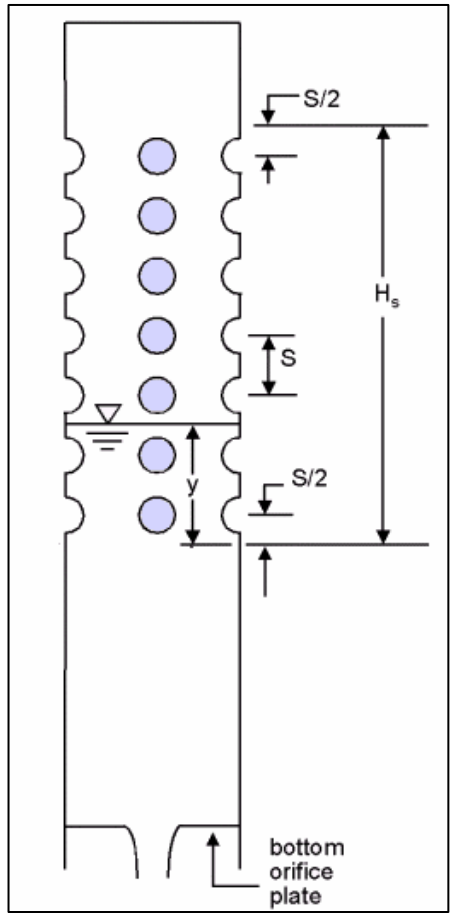


Figure 4.6-3 Perforated Riser

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$Q = 0.6A\,(2gH)^{0.5} = 3.78D^2H^{0.5} \qquad (4.6.2)$$

where:

D = diameter of orifice or pipe (ft)

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 4.6-2(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 4.6-2 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 4.6-2 Circular Perforation Sizing

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in^2)		
		1 column	2 columns	3 columns
1/4	1	0.05	0.1	0.15
5/16	2	0.08	0.15	0.23
3/8	2	0.11	0.22	0.33
7/16	2	0.15	0.3	0.45
1/2	2	0.2	0.4	0.6
9/16	3	0.25	0.5	0.75
5/8	3	0.31	0.62	0.93
11/16	3	0.37	0.74	1.11
3/4	3	0.44	0.88	1.32
13/16	3	0.52	1.04	1.56
7/8	3	0.6	1.2	1.8
15/16	3	0.69	1.38	2.07
1	4	0.79	1.58	2.37
1 1/16	4	0.89	1.78	2.67
1 1/8	4	0.99	1.98	2.97
1 3/16	4	1.11	2.22	3.33
1 1/4	4	1.23	2.46	3.69
1 5/16	4	1.35	2.7	4.05
1 3/8	4	1.48	2.96	4.44
1 7/16	4	1.62	3.24	4.86
1 1/2	4	1.77	3.54	5.31
1 9/16	4	1.92	3.84	5.76
1 5/8	4	2.07	4.14	6.21
1 11/16	4	2.24	4.48	6.72
1 3/4	4	2.41	4.82	7.23
1 13/16	4	2.58	5.16	7.74
1 7/8	4	2.76	5.52	8.28
1 15/16	4	2.95	5.9	8.85
2	4	3.14	6.28	9.42
Number of columns refers to parallel columns of holes				
Minimum plate thickness		1/4”	5/16”	3/8”

Source: Urban Drainage and Flood Control District, Denver, CO

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 4.6-4 provides a schematic of an orifice plate outlet structure for a wet extended detention pond showing the design pool elevations and the flow control mechanisms.

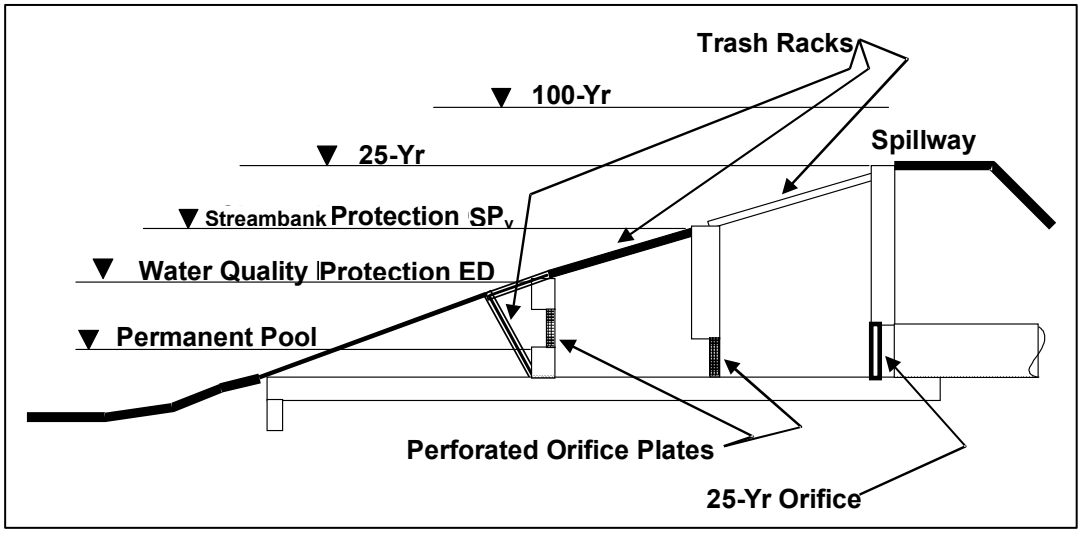


Figure 4.6-4 Schematic of Orifice Plate Outlet Structure

4.6.2.4 Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 4.6-3. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 4.6-3, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s}\sqrt{2g}H^{3/2} \qquad \textbf{(4.6.3)}$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft^2)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

4.6.2.5 Pipes and Culverts

Discharge pipes are often used as outlet structures for storm water control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or streambank protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls (see subsection 4.6.2.6). As the stage increases the flow will transition to orifice flow.

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 4.3, *Culvert Design*, or by using equation 4.6.4 (NRCS, 1984).

The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH) / (1 + k_m + k_pL)]0.5 \qquad (4.6.4)$$

where:

- Q = discharge (cfs)
- a = pipe cross sectional area (ft^2)
- g = acceleration of gravity (ft/s^2)
- H = elevation head differential (ft)
- k_m = coefficient of minor losses (use 1.0)
- k_p = pipe friction coefficient = $5087n^2/D^{4/3}$
- L =pipe length (ft)

4.6.2.6 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested* weir. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with compensation for end contractions is illustrated in Figure 4.6-5(a). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_C)]\ LH^{1.5} \qquad (4.6.5)$$

were:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

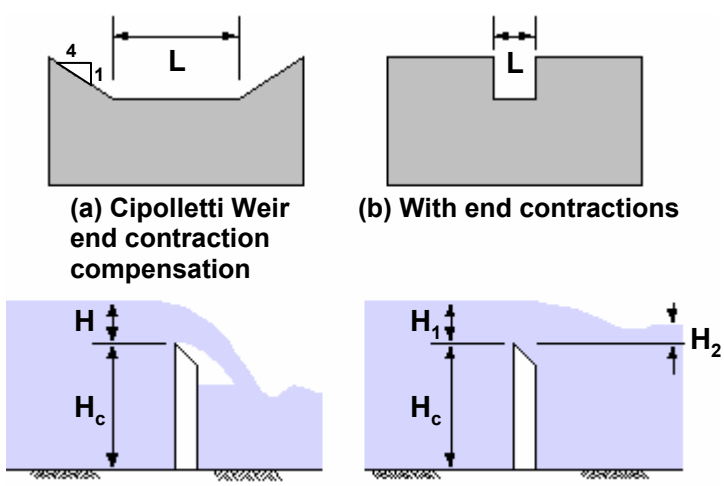


Figure 4.6-5 Sharp-Crested Weir

The discharge equation for the Cipolletti Weir is $Q = 3.367\ LH^{1/2}$

A sharp-crested weir with two end contractions is illustrated in Figure 4.6-5(b). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_C)]\ (L - 0.2H)\ H^{1.5} \qquad (4.6.6)$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_S = Q_f\,(1 - (H_2/H_1)^{1.5})^{0.385} \qquad (4.6.7)$$

where:

- Q_S = submergence flow (cfs)
- Q_f = free flow (cfs)
- H_1 = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

4.6.2.7 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad (4.6.8)$$

where:

- Q = discharge (cfs)
- C = broad-crested weir coefficient
- L = broad-crested weir length perpendicular to flow (ft)
- H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 4.6-3.

Figure 4.6-6
Broad-Crested Weir

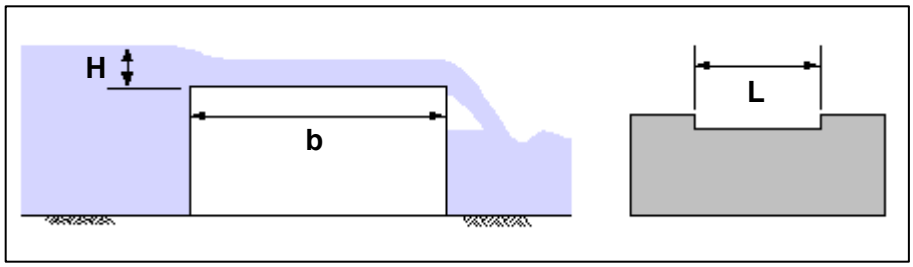


Table 4.6-3 Broad-Crested Weir Coefficient (C) Values

Measured Head (H)*	Weir Crest Breadth (b) in feet										
In feet	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

* Measured at least 2.5H upstream of the weir.
Source: Brater and King (1976)

4.6.2.8 V-Notch Weirs

The discharge through a V-notch weir (Figure 4.6-7) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)\, H^{2.5} \qquad (4.6.9)$$

where:

Q = discharge (cfs)
θ = angle of V-notch (degrees)
H = head on apex of notch (ft)

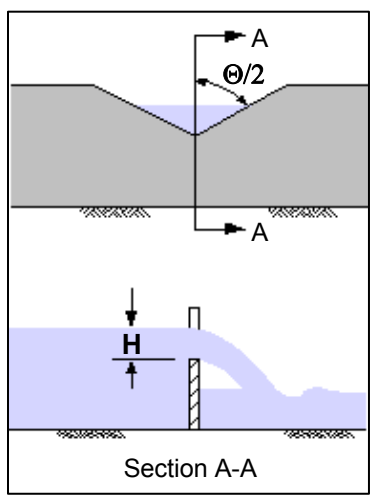


Figure 4.6-7 V-Notch Weir

4.6.2.9 Proportional Weirs

Although it may be more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 4.6-8. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\, a^{0.5}\, b\, (H - a/3) \qquad (4.6.10)$$

$$x/b = 1 - (1/3.17)\, (\arctan (y/a)^{0.5}) \qquad (4.6.11)$$

where:

Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 4.6-8

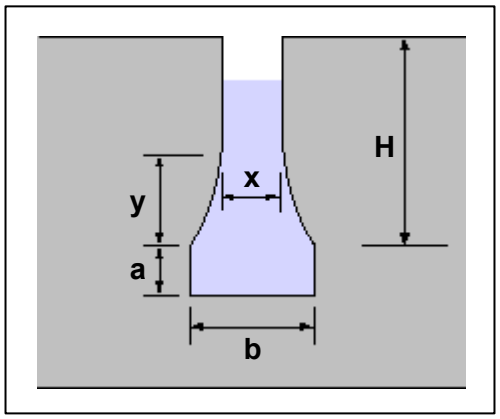


Figure 4.6-8 Proportional Weir Dimensions

4.6.2.10 Combination Outlets

Combinations of orifices, weirs, and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality protection volume, streambank protection volume, and flood control volume).

They are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 4.6-9 shows an example of a riser designed for a wet extended detention pond. The orifice plate outlet structure in Figure 4.6-4 is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 4.6-10) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

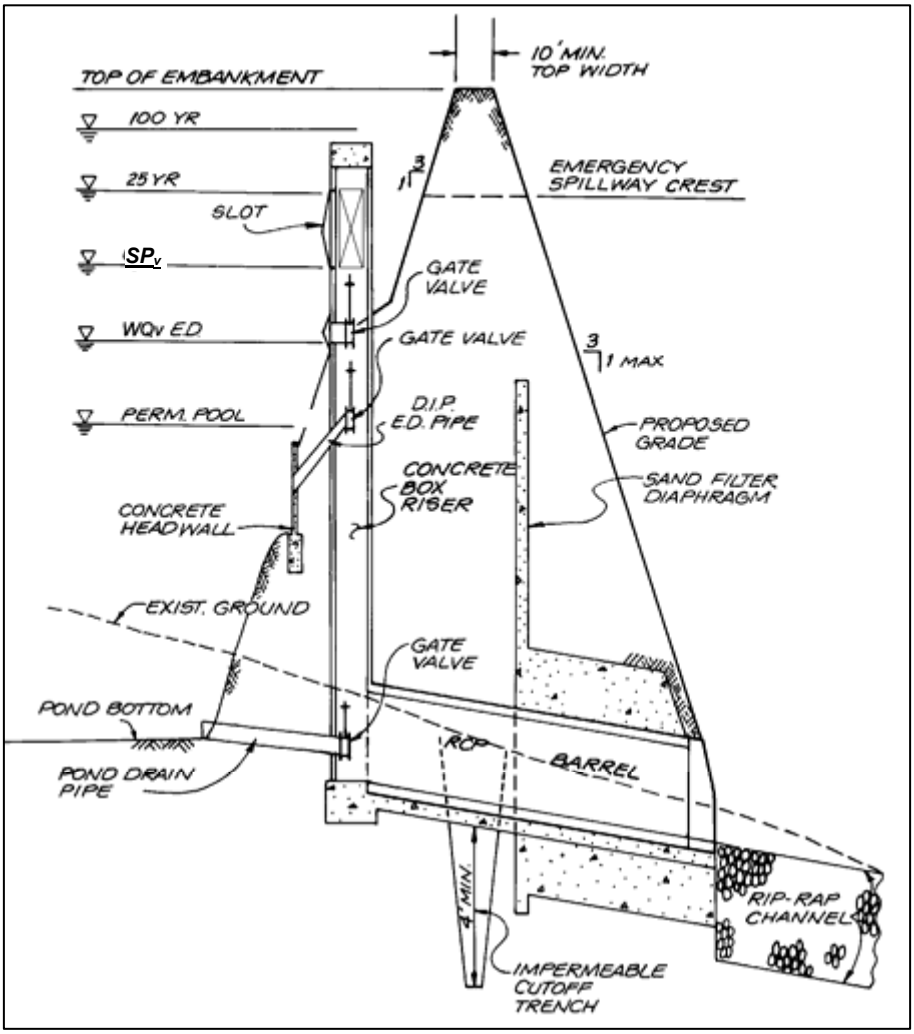


Figure 4.6-9 Schematic of Combination Outlet Structure

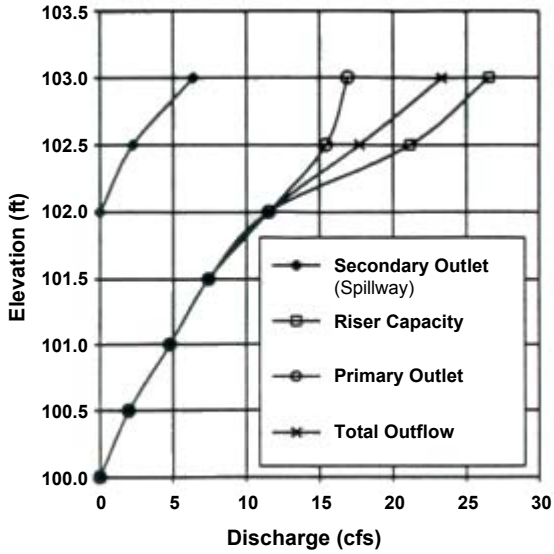


Figure 4.6-10 Composite Stage-Discharge Curve

4.6.3 Extended Detention (Water Quality and Streambank Protection) Outlet Design

4.6.3.1 Introduction

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for downstream streambank protection or the ED portion of the water quality protection volume. The release rate for both the WQ_v and SP_v should be one that discharges the ED volume in a period of 24 hours or longer. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQ_v extended detention and SP_v control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the streambank protection drawdown.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

The outlet hydraulics for peak control design (flood control) is usually straightforward in that an outlet is selected to limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality protection or downstream streambank protection, however, the storage volume is detained and released for each over a specified amount of time (e.g., 24-hours). The release period is a "brim" drawdown time, beginning at the time of peak storage of the WQ_v or SP_v until the entire calculated volume drains out of the basin. This assumes the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- Using the maximum hydraulic head associated with the brim storage volume and maximum discharge, calculate the orifice size needed to achieve the required drawdown time. Route the volume through the basin to verify the actual storage volume used and the drawdown time.
- Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or streambank protection.

4.6.3.2 Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality protection volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality protection design.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1 Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Protection Volume (or Streambank Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs
$Q_{max} = 2 * Q_{avg} = 2 * 0.38 = 0.76$ cfs

Step 2 Determine the required orifice diameter by using the orifice equation (4.6.8) and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

Determine pipe diameter from $A = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.071)/3.14]^{0.5} = 0.30\ ft = 3.61\ in$

Use a 3.6-inch diameter water quality protection orifice.

Routing the water quality protection volume of 0.76 ac ft through the 3.6-inch water quality protection orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours.

4.6.3.3 Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example (4.6.3.2) use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Average Hydraulic Head (h_{avg}) = 2.5 ft (from stage vs storage data)

Step 1 Determine the average release rate to release the water quality protection volume over a 24-hour time period.

$Q = 33{,}106\ ft^3 / (24\ hr)(3{,}600\ s/hr) = 0.38\ cfs$

Step 2 Determine the required orifice diameter by using the orifice equation (4.6.8) and the average head on the orifice:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05\ ft^3$

Determine pipe diameter from $A = 3.14r^2 = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.05)/3.14]^{0.5} = 0.252\ ft = 3.03\ in$

Use a 3-inch diameter water quality protection orifice.

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

4.6.4 Multi-Stage Outlet Design

4.6.4.1 Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality protection, streambank protection, and flood control) for storm water ponds, storm water wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge

from a facility during multiple design storms. Figures 4.6-4 and 4.6-9 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 4.6-10)

4.6.4.2 Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQ_v, SP_v, and Q_f), then that step in the procedure is skipped.

Step 1 Determine Storm Water Control Volumes. Using the procedures from Sections 2.1 and 2.2, estimate the required storage volumes for water quality protection (WQ_v), streambank protection (SP_v), and flood control (Q_f).

Step 2 Develop Stage-Storage Curve. Using the structure geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

Step 3 Design Water Quality Protection Outlet. Design the water quality protection extended detention (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in subsection 4.6.3. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality protection will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see subsection 4.6.5).

Step 4 Design Streambank Protection Outlet. Design the streambank protection extended detention outlet (SP_v-ED) using either method from subsection 4.6.3. For this design, the storage needed for streambank protection will be greater than the water quality protection volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality protection orifice and the outlet used for streambank protection. The outlet should be protected in a manner similar to that for the water quality protection orifice.

Step 5 Design Flood Control Outlet. The storage needed for flood control will be greater than the water quality protection and streambank protection storage. Establish the Q_f maximum water surface elevation using the stage-storage curve and subtract the SP_v elevation to find the maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment peak discharge rate. Develop a stage-discharge curve for the combined set of outlets (WQ_v, SP_v and Q_f).

Step 6 Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the outlet structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 4.6-11, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this transition will occur. Also note in Figure 4.6 -11 that as the elevation of the water increases further, the

control can change from barrel inlet flow control to barrel pipe flow control. Figure 4.6-12 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

Step 7 Size the Emergency Spillway. It is recommended that all storm water impoundment structures have a vegetated emergency spillway (see subsection 4.6.7). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed. Also check the dam safety requirements to be sure of an adequate design.

Step 8 Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See Subsection 4.7, *Energy Dissipation Design*, for more information.

Step 9 Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.

Step 10 Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

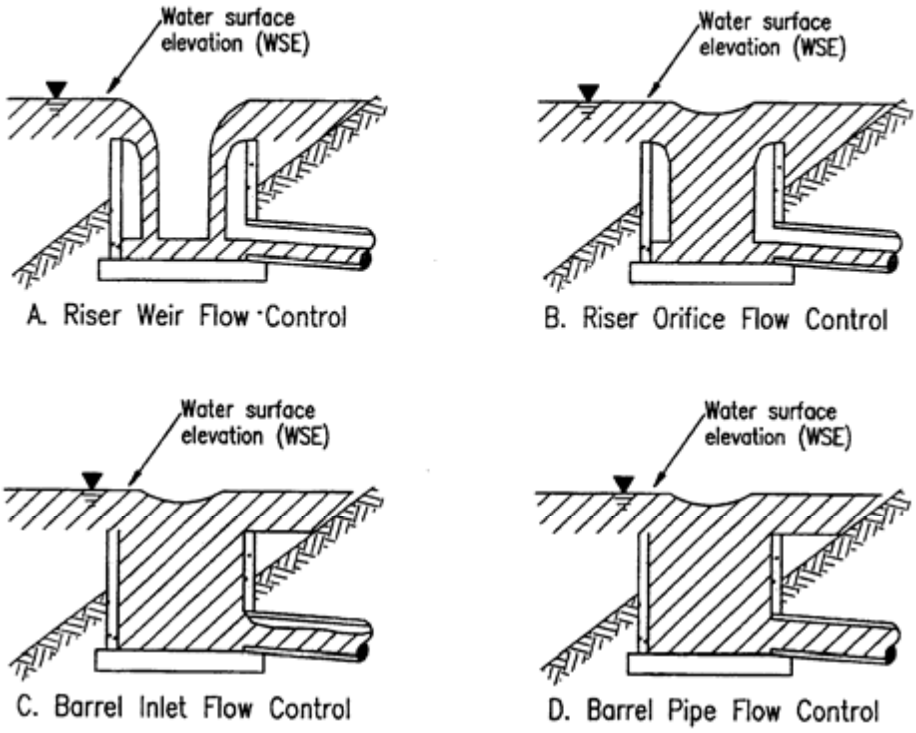


Figure 4.6-11 Riser Flow Diagrams
(Source: VDCR, 1999)

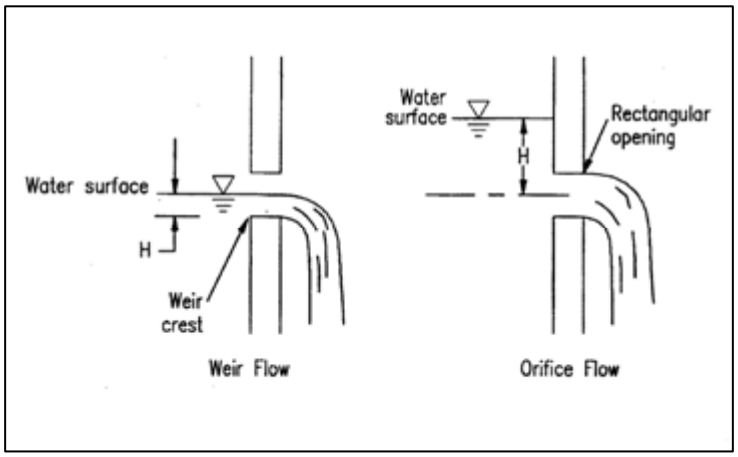


Figure 4.6-12 Weir and Orifice Flow
(Source: VDCR, 1999)

4.6.5 Extended Detention Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

The use of a reverse slope pipe attached to a riser for a storm water pond or wetland with a permanent pool (see Figure 4.6-13). The inlet is submerged a minimum of 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.

The use of a hooded outlet for a storm water pond or wetland with a permanent pool (see Figures 4.6-14 and 4.6-15).

Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 4.6-16).

Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

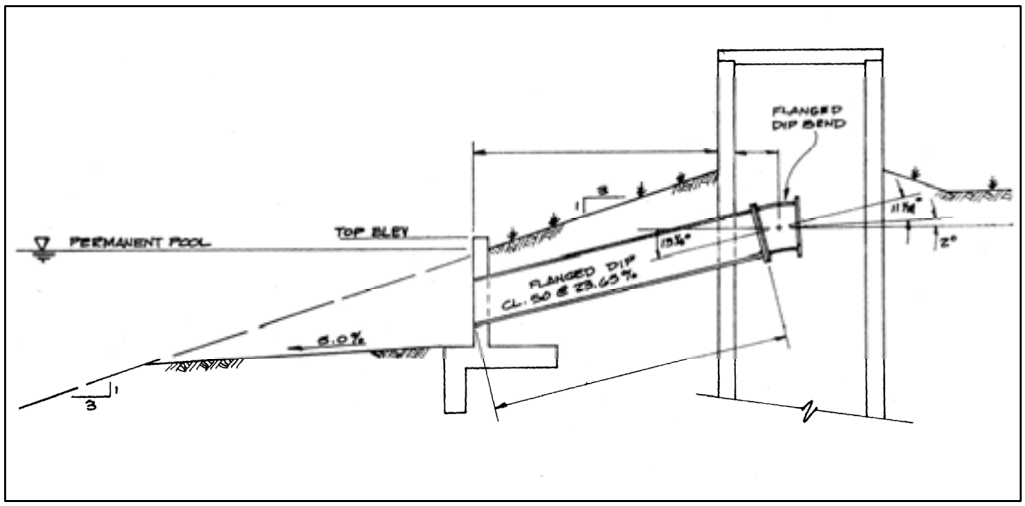


Figure 4.6-13 Reverse Slope Pipe Outlet

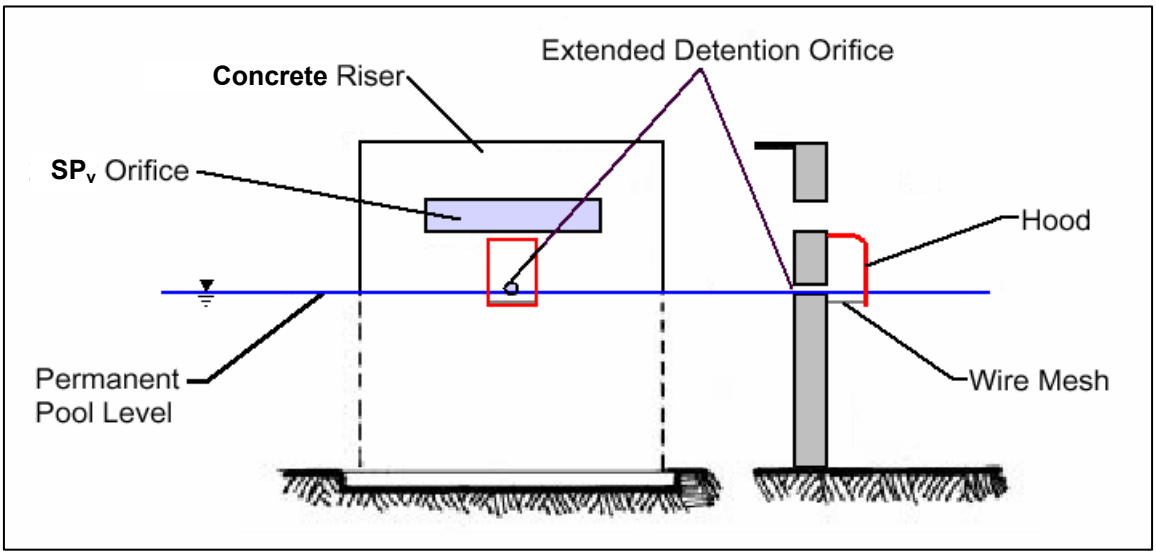


Figure 4.6-14 Hooded Outlet

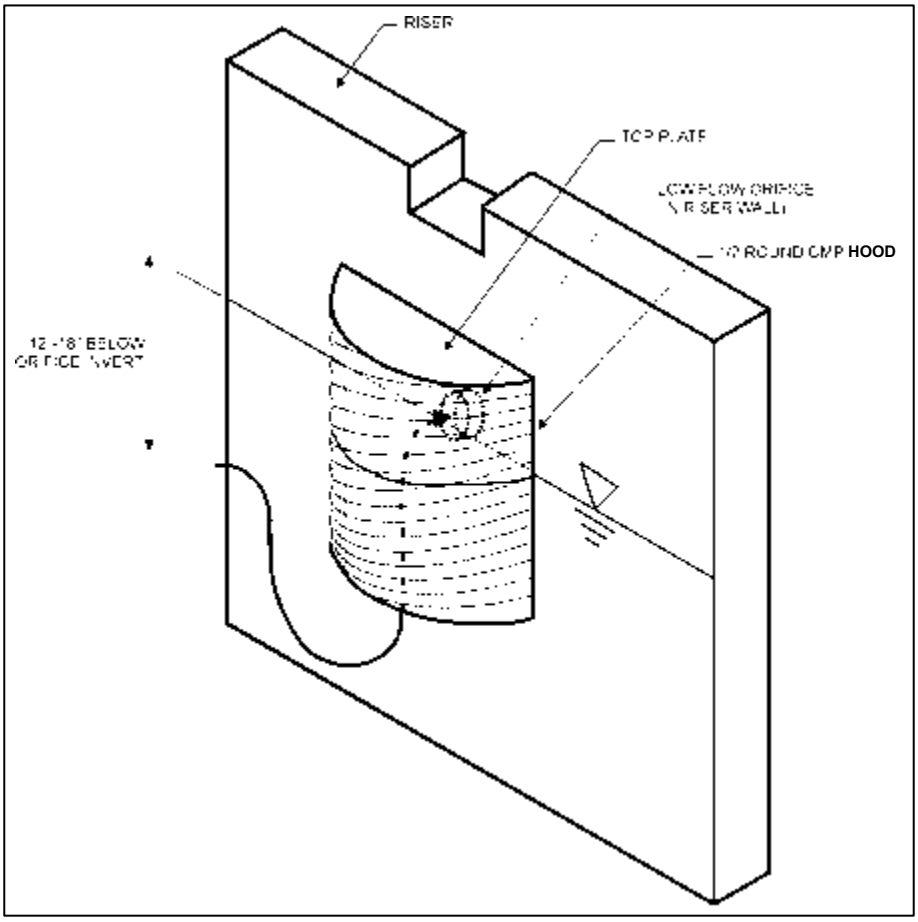


Figure 4.6-15 Half-Round CMP Orifice Hood

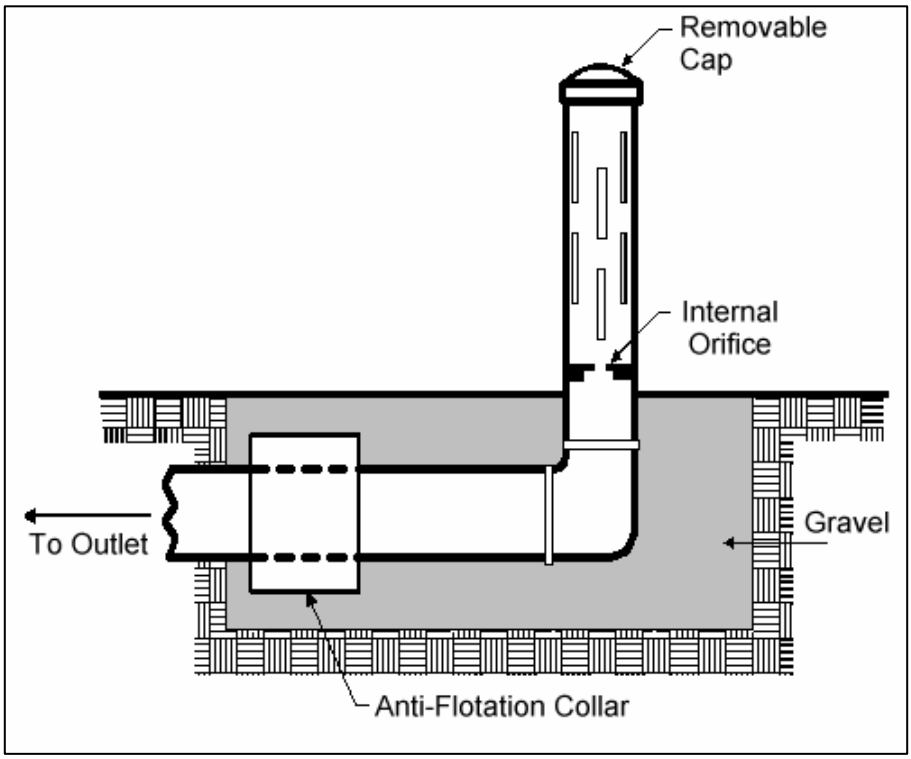


Figure 4.6-16 Internal Control for Orifice Protection

4.6.6 Trash Racks and Safety Grates

4.6.6.1 Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure 4.6-17. Additional track rack design can be found in Appendix C. The inclined vertical bar rack is most effective for lower stage

outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

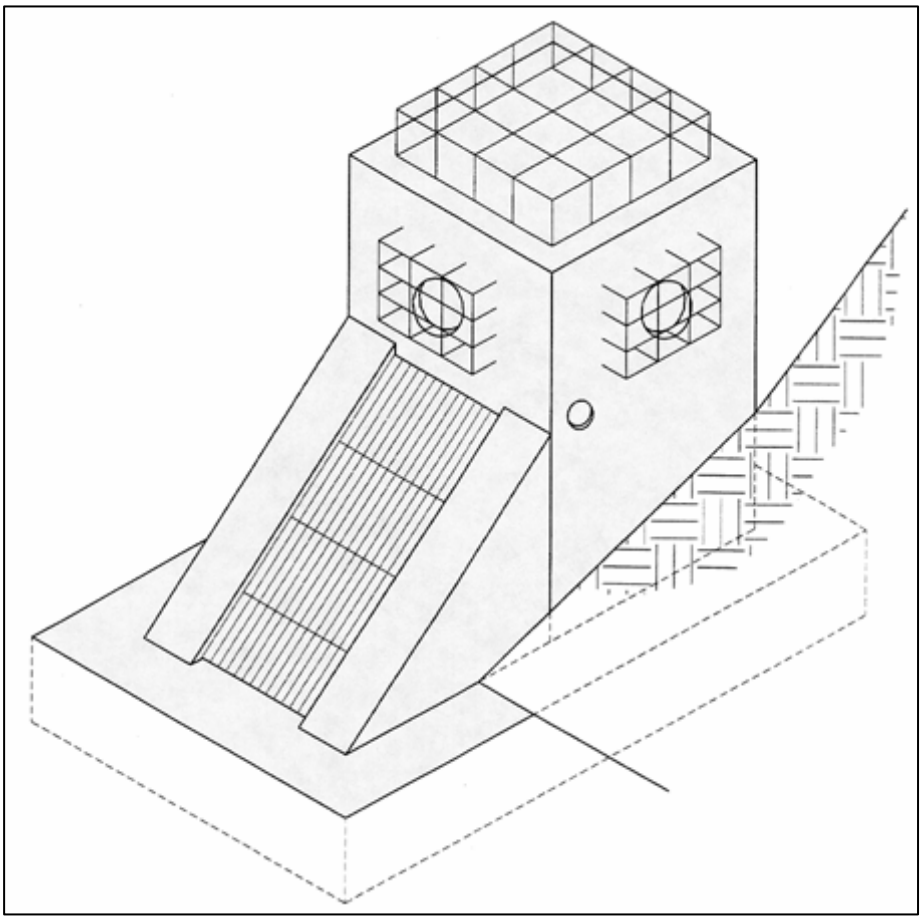


Figure 4.6-17 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

4.6.6.2 Trash Rack Design

Trash racks must be large enough so that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level — the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 4.6-18 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$H_g = K_{g1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g \qquad (4.6.12)$$

Where:

- H_g = head loss through grate (ft)
- K_{g1} = bar shape factor:
  - 2.42 - sharp edged rectangular
  - 1.83 - rectangular bars with semicircular upstream faces
  - 1.79 - circular bars
  - 1.67 - rectangular bars with semicircular up- and downstream faces
- w = maximum cross-sectional bar width facing the flow (in)
- x = minimum clear spacing between bars (in)
- V_u = approach velocity (ft/s)
- g = acceleration due to gravity (32.2 ft/s^2)
- θ_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$H_g = \frac{K_{g2} V_u^2}{2g} \qquad (4.6.13)$$

Where:

K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)

$K_{g2} = 0.00158 - 0.03217\ A_r + 7.1786\ A_r^2$

- sharp edged rectangular (length/thickness = 5)
  $K_{g2} = -0.00731 + 0.69453\ A_r + 7.0856\ A_r^2$
- round edged rectangular (length/thickness = 10.9)
  $K_{g2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- circular cross section
  $K_{g2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

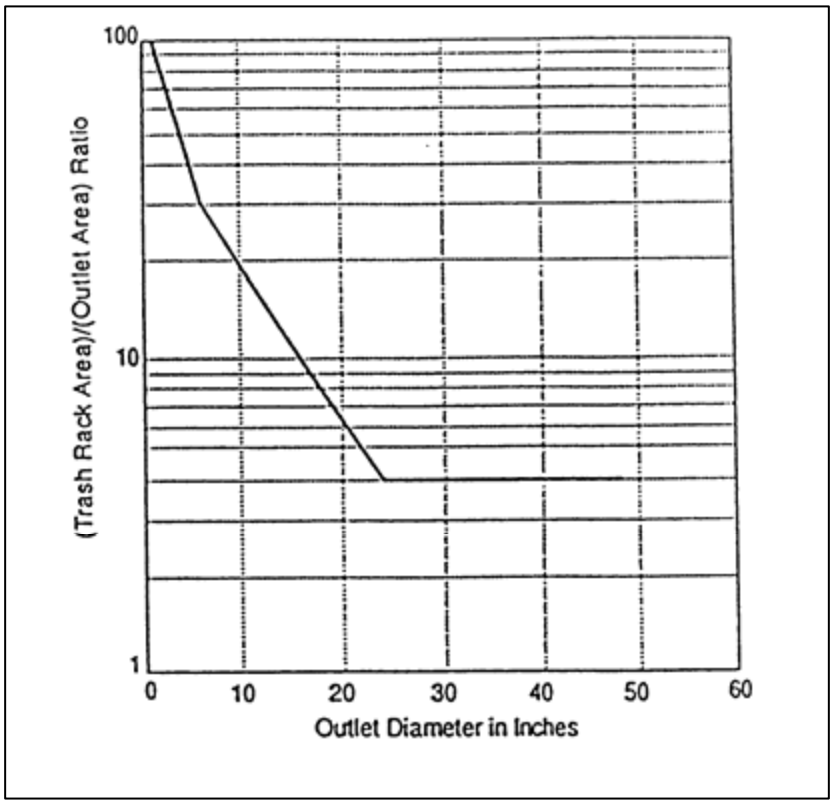


Figure 4.6-18 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

4.6.7 Secondary Outlets

4.6.7.1 Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 4.6-19 shows an example of an emergency spillway.

In many cases, on-site storm water storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

4.6.7.2 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 4.6-19). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Any dam, six feet or higher, must meet appropriate state and federal design standards, especially those regarding spillway design requirements related to passage of the probable maximum flood. In any case, the 100-year flood discharge, assuming blockage of outlet works, must be conveyed with some freeboard as specified by local criteria. Flow in the emergency spillway is open channel flow (see Section 4.4, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

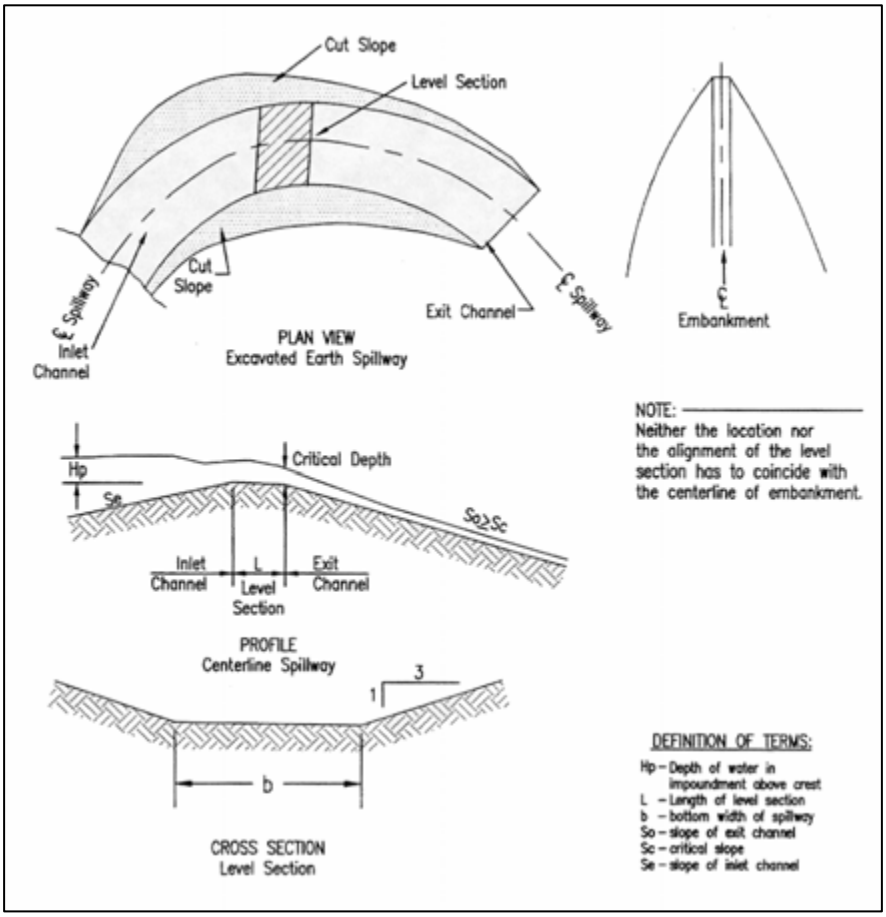


Figure 4.6-19 Emergency Spillway
(Source: VDCR, 1999)

References

Allred-Coonrod, J. E., 1991, Safety Grates in Open Channels, MS Prof. Paper, University of New Mexico, Albuquerque.

Brater, E. F. and H. W. King, 1976. Handbook of Hydraulics. 6th ed. New York: McGraw Hill Book Company.

Bureau of Reclamation. Flood Hydrology Manual, A Water Resources Technical Publication.

Chow, C. N., 1959. Open Channel Hydraulics. New York: McGraw Hill Book Company.

Debo, Thomas N and Andrew J. Reese, 2003. Municipal Storm Water Management. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

McEnroe, B.M., J.M. Steichen and R. M. Schweiger, 1988. Hydraulics of Perforated Riser Inlets for Underground Outlet Terraces, Trans ASAE, Vol. 31, No. 4, 1988.

Metcalf & Eddy, Inc., 1972, Wastewater Engineering, McGraw-Hill, New York.

NRCS, 1984. Engineering Field Manual for Conservation Practices, Soil Conservation Service, Engineering Division, Washington, D.C.

Virginia Department of Conservation and Recreation, 1999. Virginia Stormwater Management Handbook.

Sandvik, A., 1985. Proportional Weirs for Stormwater Pond Outlets. Civil Engineering, March 1985, ASCE pp. 54-56.

Metropolitan Government of Nashville and Davidson County, 1988. Stormwater Management Manual - Volume 2 Procedures. Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill.

United States Bureau of Reclamation, Water Measurement Manual, http://www.usbr.gov/wrrl/fmt/wmm/

U.S. Army Corps of Engineers Engineers Manual (EM 1110-2-1405). Flood –Hydrograph Analysis and Computations.

U.S. Army Corps of Engineers, 1988, Hydraulic Design Criteria (HDC), USAE Waterways Experiment Station, Vicksburg, MS.

Urban Drainage and Flood Control District, 1999. Criteria Manual, Denver, CO.

Wycuff, R. L. and U. P. Singh, 1976. Preliminary Hydrologic Design of Small Flood Detention Reservoirs. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49.

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Section 4.7
Energy Dissipation

4.7.1 Overview

4.7.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Storm water transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at storm water outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

Energy dissipators are engineered devices such as rip-rap aprons or concrete baffles placed at the outlet of storm water conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

4.7.1.2 General Criteria

Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosion velocity of the downstream area channel system.

Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

4.7.1.3 Recommended Energy Dissipators

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, for the design procedures of other energy dissipators.

4.7.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.7-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.7-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
D	Height of box culvert	ft
d_{50}	Size of riprap	ft
d_w	Culvert width	ft
Fr	Froude Number	-
g	Acceleration of gravity	ft/s^2
h_s	Depth of dissipator pool	ft
L	Length	ft
L_a	Riprap apron length	ft
L_B	Overall length of basin	ft
L_s	Length of dissipator pool	ft
PI	Plasticity index	-
Q	Rate of discharge	cfs
S_v	Saturated shear strength	lbs/in^2
t	Time of scour	min.
t_c	Critical tractive shear stress	lbs/in^2
TW	Tailwater depth	ft
V_L	Velocity L feet from brink	ft/s
V_o	Normal velocity at brink	ft/s
V_o	Outlet mean velocity	ft/s
V_s	Volume of dissipator pool	ft^2
W_o	Diameter or width of culvert	ft
W_s	Width of dissipator pool	ft
y_e	Hydraulic depth at brink	ft
y_o	Normal flow depth at brink	ft

4.7.3 Design Guidelines

If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

If outlet protection is not provided, energy dissipation will occur through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, h_s. The scourhole should then be stabilized. If the scourhole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 4.7-1 provides the riprap size recommended for use downstream of energy dissipators.

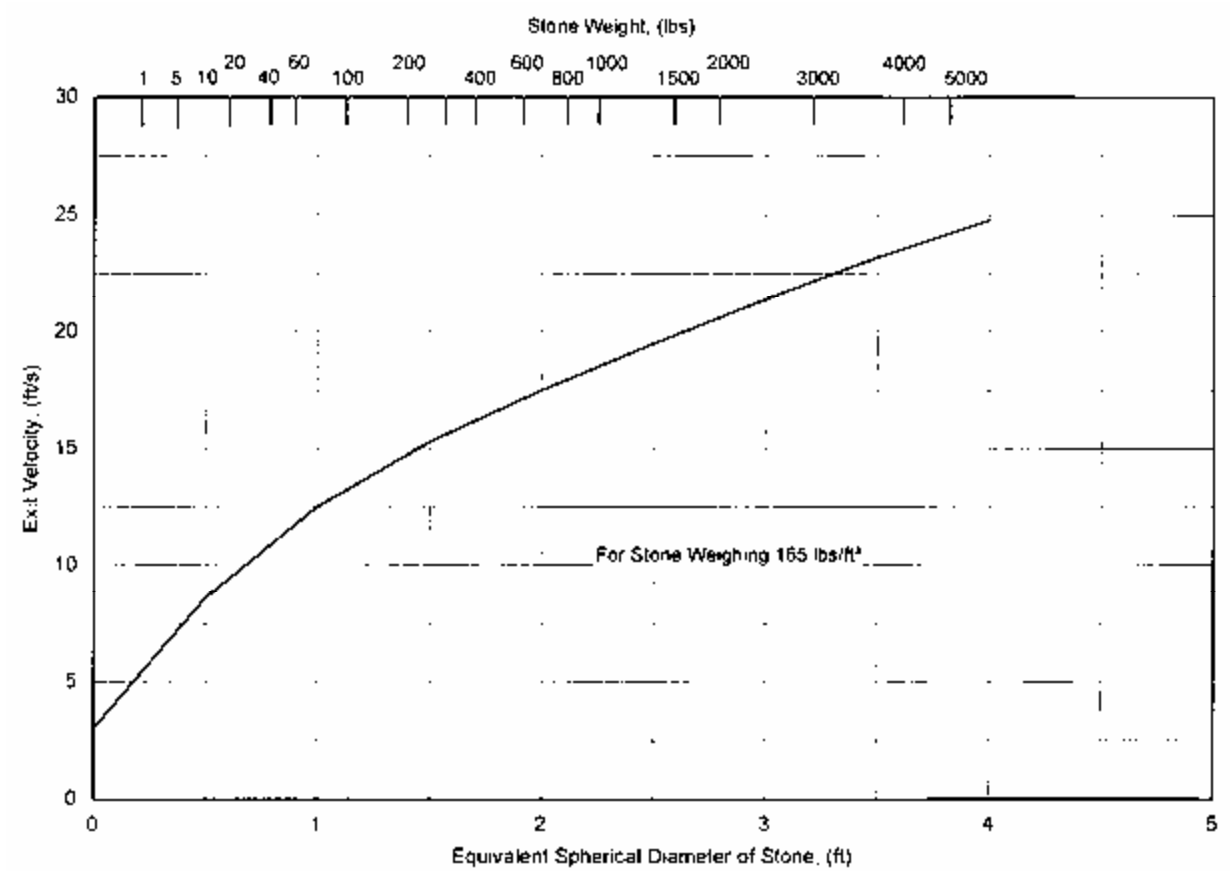


Figure 4.7-1 Riprap Size for Use Downstream of Energy Dissipator
(Source: Searcy, 1967)

4.7.4 Riprap Aprons

4.7.4.1 Description

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.

4.7.4.2 Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 4.7-2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 4.7-3 should be used.

Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 4.7-2 and 4.7-3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 4.7-4.

a. For pipes flowing full:

   Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.

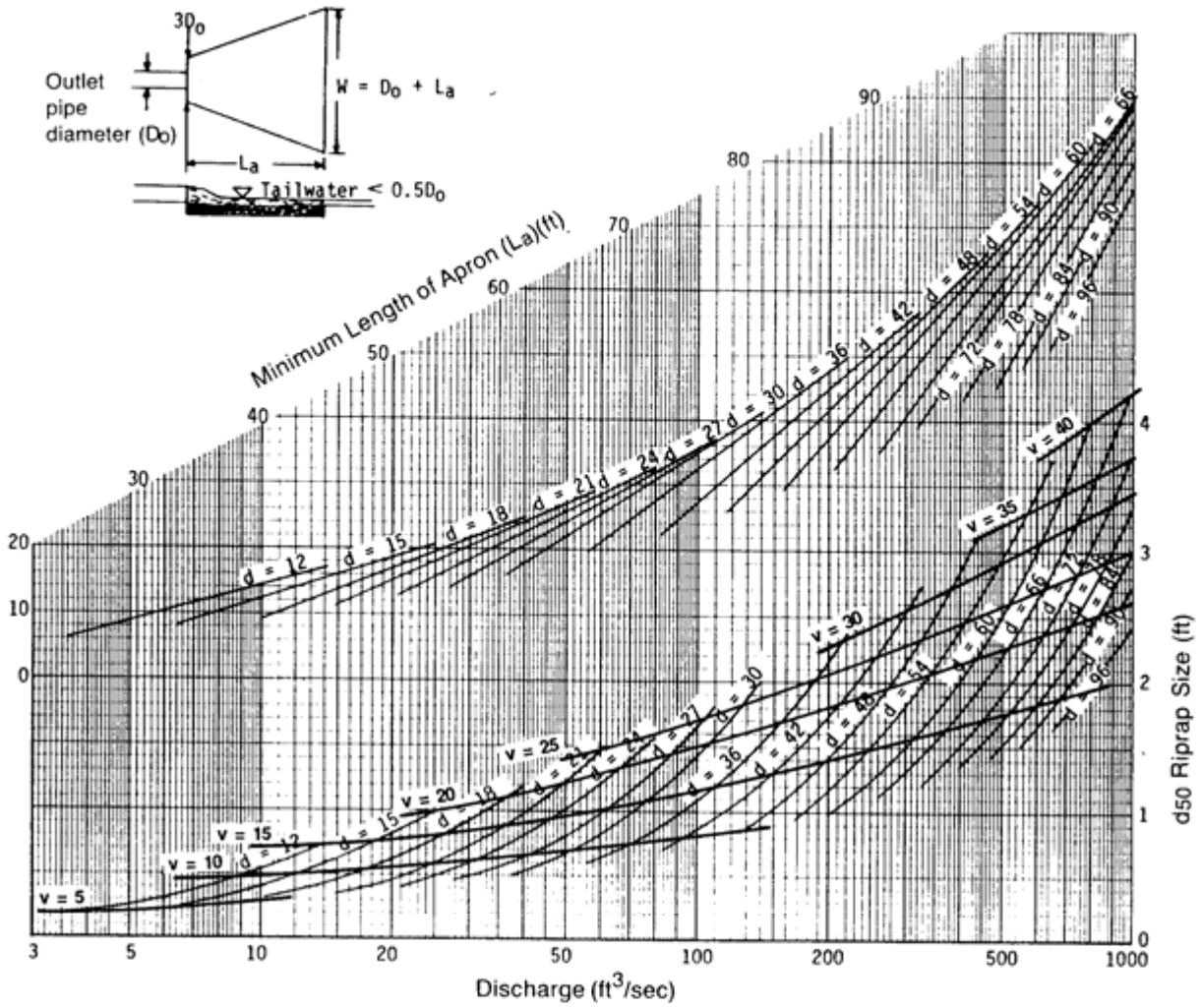
b. For pipes flowing partially full:

   Use the depth of flow, d, in feet, and velocity, v, in ft/s. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, L_a, from the scale on the left.

c. For box culverts:

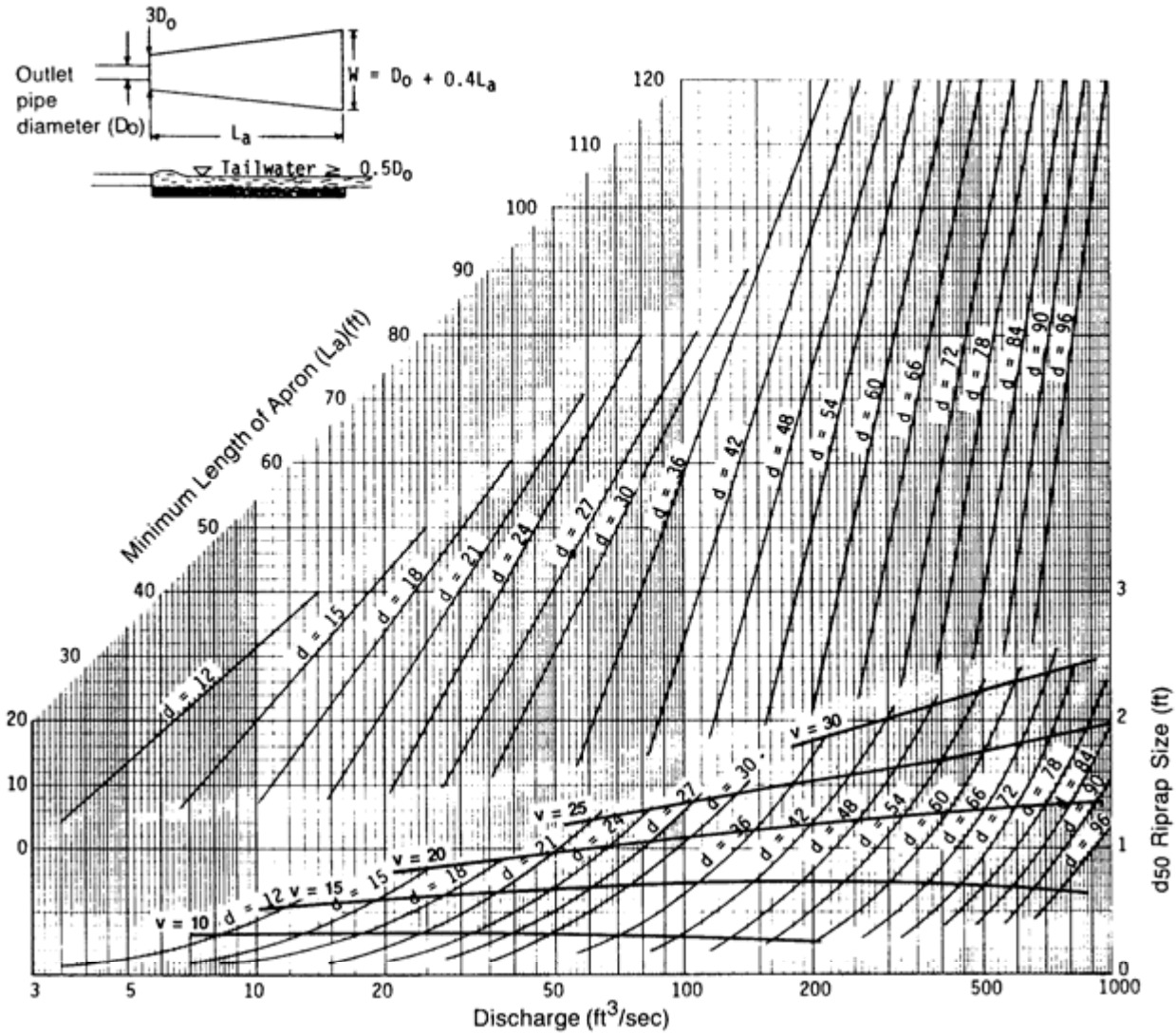
   Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 4.7-4. This will provide protection under either of the tailwater conditions.



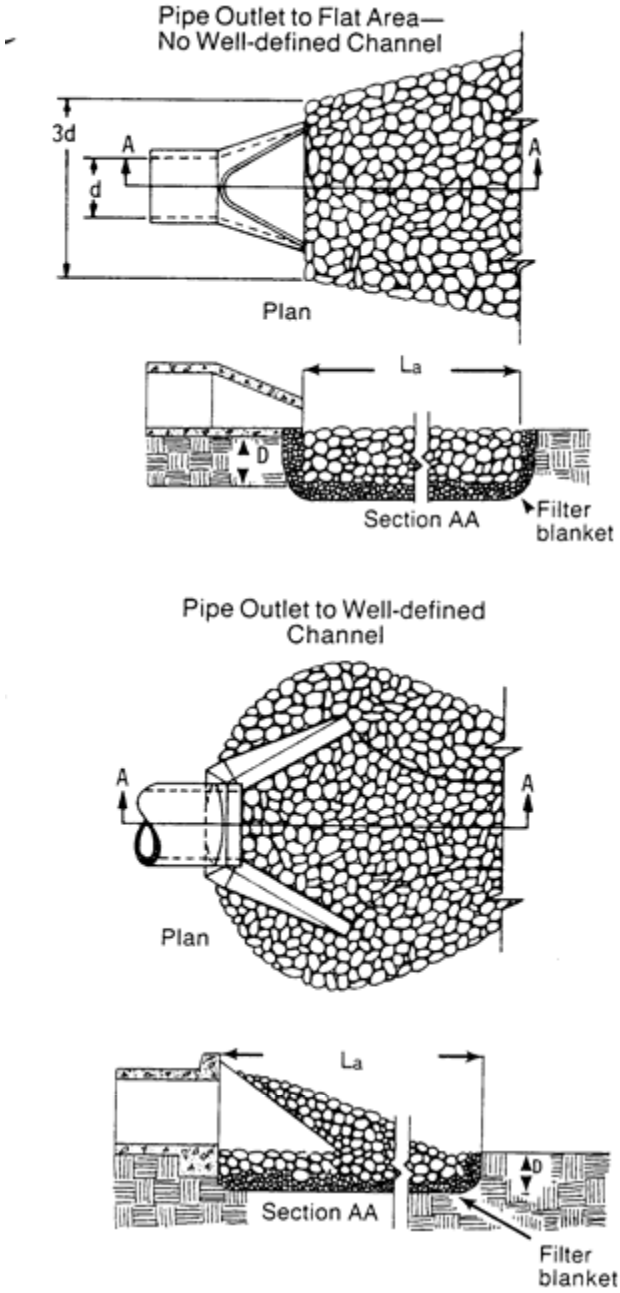
Curves may not be extrapolated.

Figure 4.7-2 Design of Riprap Apron under Minimum Tailwater Conditions
(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.7-3 Design of Riprap Apron under Maximum Tailwater Conditions
(Source: USDA, SCS, 1975)



Notes

1. L_a is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter but not less than 6".
3. In a well-defined channel extend the apron up the channel banks to an elevation of 6" above the maximum tailwater depth or to the top of the bank, whichever is less.
4. A filter blanket or filter fabric should be installed between the riprap and soil foundation.

Figure 4.7-4 Riprap Apron
(Source: Manual for Erosion and Sediment Control in Georgia, 1996)

4.7.4.3 Design Considerations

The following items should be considered during riprap apron design:

The maximum stone diameter should be 1.5 times the median riprap diameter.
d_{max} = 1.5 x d_{50} , d_{50} = the median stone size in a well-graded riprap apron.

The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater.
Apron thickness = 1.5 x d_{max}
(Apron thickness may be reduced to 1.5 x d_{50} when an appropriate filter fabric is used under the apron.)

The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.

If there is a well-defined channel, the apron length should be extended as necessary so the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.

If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.

The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

4.7.4.4 Example Designs

<u>Example 1</u> Riprap Apron Design for Minimum Tailwater Conditions

A flow of 280 cfs discharges from a 66-in pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

Minimum tailwater conditions = 0.5 d_o, d_o = 66 in = 5.5 ft; therefore, 0.5 d_o = 2.75 ft.

Since TW = 2 ft is less than 2.75 ft, use Figure 4.7-2 for minimum tailwater conditions.

By Figure 4.7-2, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

The downstream apron width equals the apron length plus the pipe diameter:

$W = d + L_a = 5.5 + 38 = 43.5$ ft

Maximum riprap diameter is 1.5 times the median stone size:

$1.5\ (d_{50}) = 1.5\ (1.2) = 1.8$ ft

Riprap depth = 1.5 (d_{max}) = 1.5 (1.8) = 2.7 ft.

Example 2 Riprap Apron Design for Maximum Tailwater Conditions

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

Minimum tailwater conditions = 0.5 d_o, d_o = 0.5 (5.0) = 2.5 ft.

Since TW = 5.0 ft is greater than 2.5 ft, use Figure 4.7-3 for maximum tailwater conditions.

$v = Q/A = 600/[(5)(10)] = 12$ ft/s

On Figure 4.7-3, at the intersection of the curve, d_o = 60 in and v = 12 ft/s, d_{50} = 0.4 ft. Reading up to the intersection with d = 60 in, find L_a = 40 ft.

Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4\ (40) = 26$ ft.

Maximum stone diameter = 1.5 d_{50} = 1.5 (0.4) = 0.6 ft.

Riprap depth = 1.5 d_{max} = 1.5 (0.6) = 0.9 ft.

4.7.5 Riprap Basins

4.7.5.1 Description

Another method to reduce the exit velocities from storm water outlets is through the use of a riprap basin. A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

4.7.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.7-5. Principal features of the basin are:

The basin is preshaped and lined with riprap of median size (d_{50}).

The floor of the riprap basin is constructed at an elevation of h_s below the culvert invert. The dimension h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge. The ratio of h_s to d_{50} of the material should be between 2 and 4.

The length of the energy dissipating pool is 10 x h_s or 3 x W_o, whichever is larger. The overall length of the basin is 15 x h_s or 4 x W_o, whichever is larger.

4.7.5.3 Design Procedure

The following procedure should be used for the design of riprap basins.

Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that $TW/y_o \leq 0.75$ for the design discharge.

For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 4.7-6 or Figure 4.7-7 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

For streambank protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 4.7-8, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

Size basin as shown in Figure 4.7-5.

Where allowable dissipator exit velocity is specified:

a. Determine the average normal flow depth in the natural channel for the design discharge.

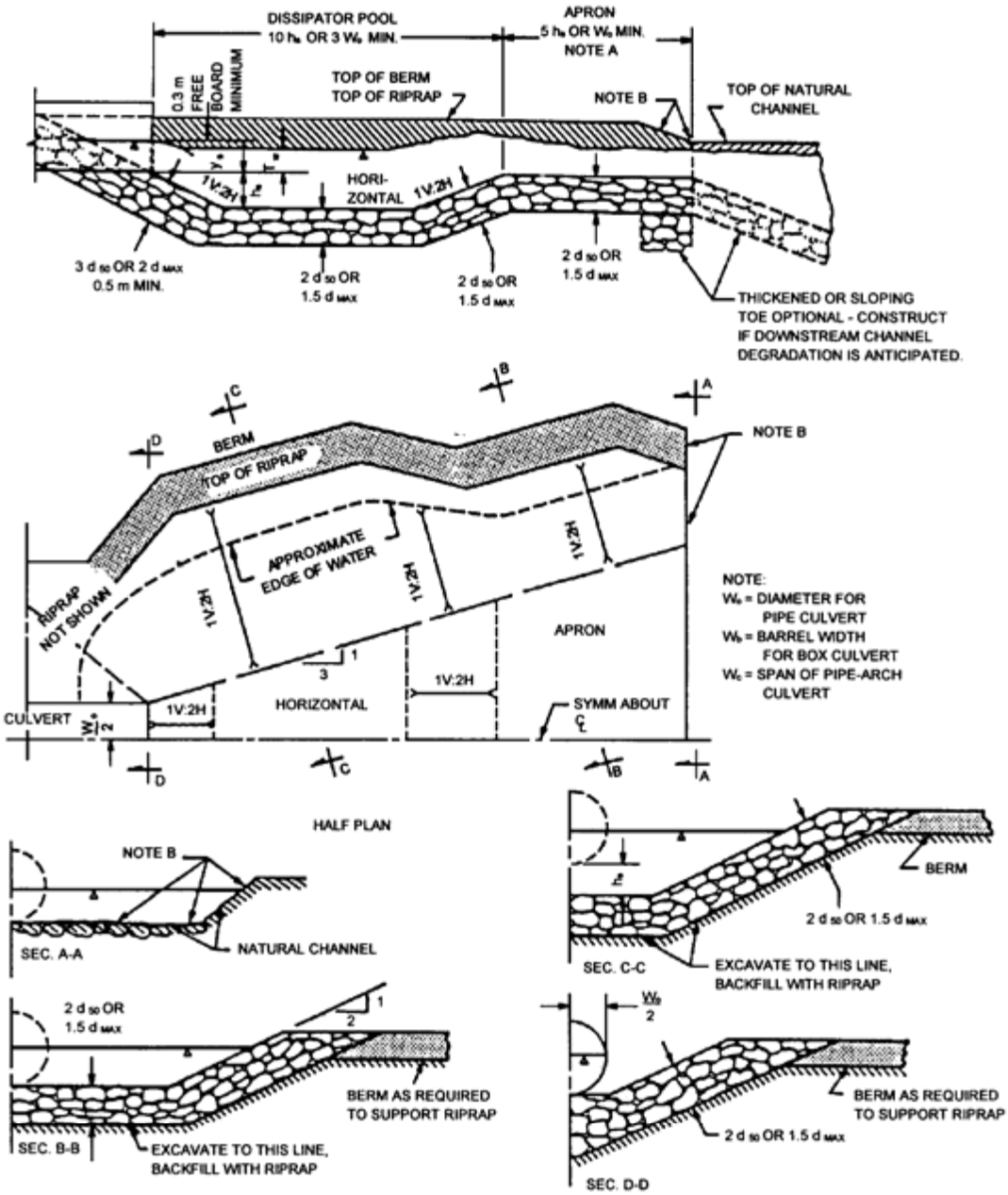
b. Extend the length of the energy basin (if necessary) so the width of the energy basin at section A-A, Figure 4.7-5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

- Design a conventional basin for low tailwater conditions in accordance with the instructions above.
- Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 4.7-9.
- Shape downstream channel and size riprap using Figure 4.7-1 and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled Use of Riprap For Bank Protection.



NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

Figure 4.7-5 Details of Riprap Outlet Basin
(Source: HEC-14, 1983)

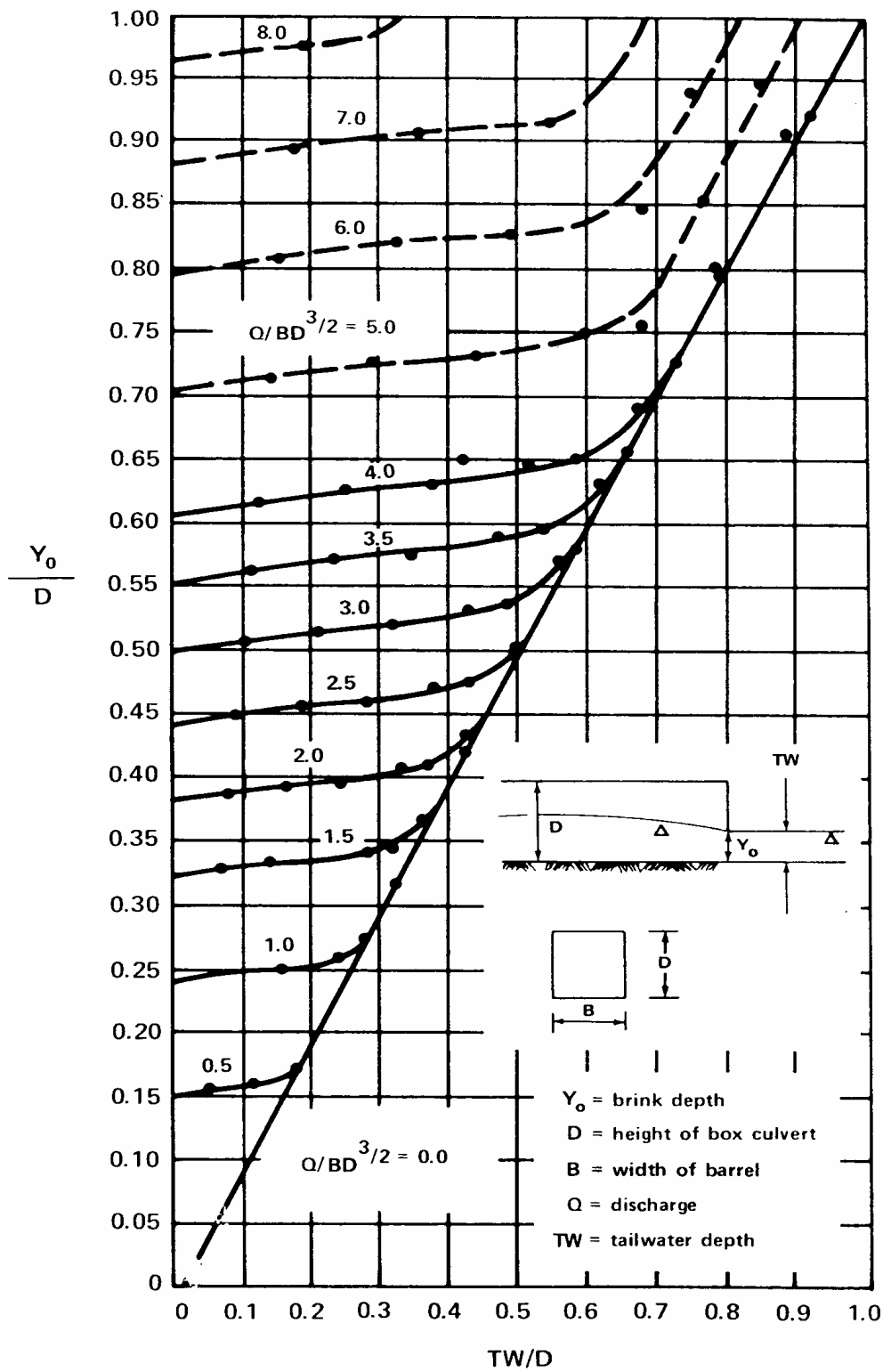


Figure 4.7-6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes
(Source: USDOT, FHWA, HEC-14, 1983)

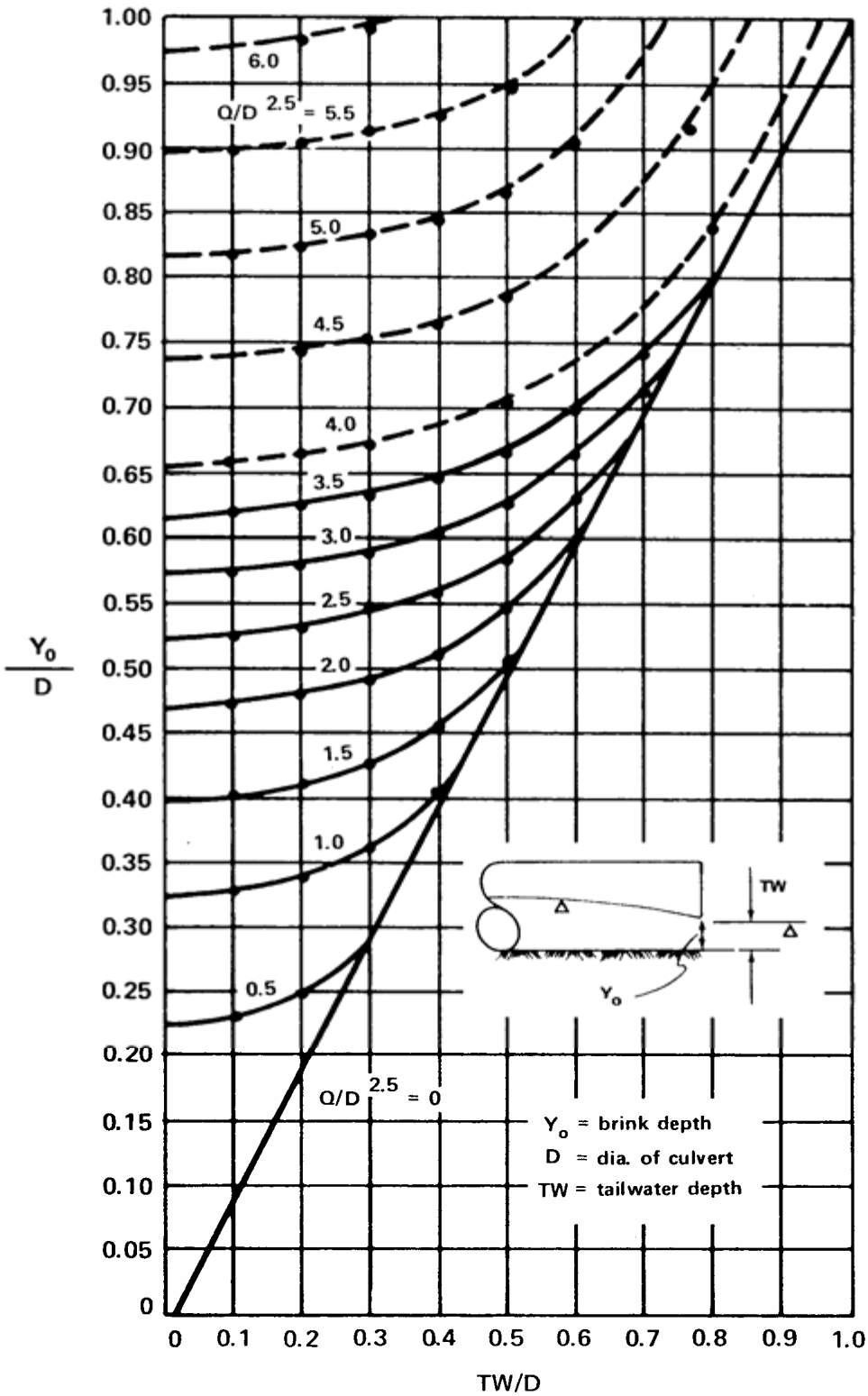


Figure 4.7-7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes
(Source: USDOT, FHWA, HEC-14, 1983)

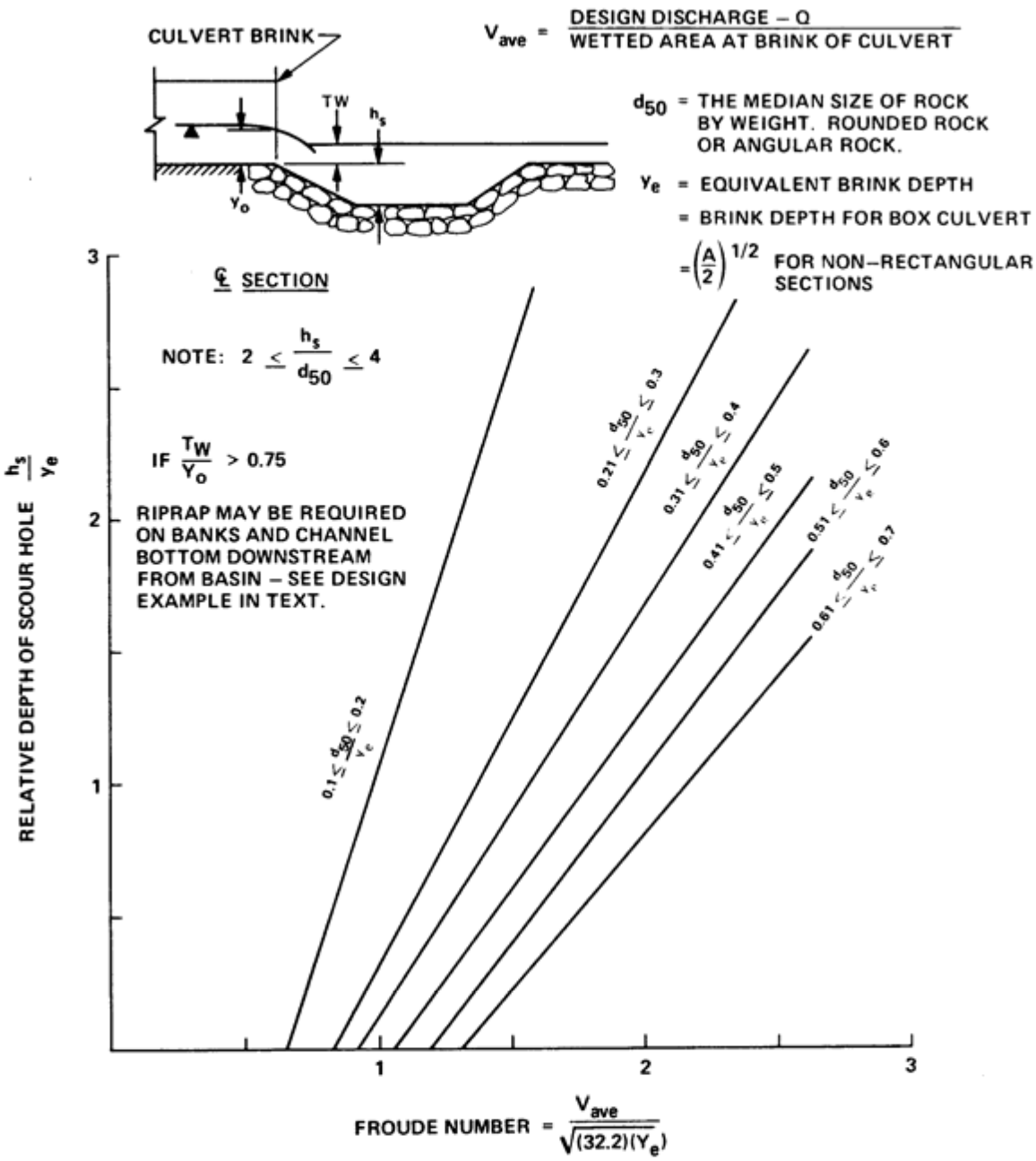


Figure 4.7-8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable
(Source: USDOT, FHWA, HEC-14, 1983)

4.7.5.4 Design Considerations

Riprap basin design should include consideration of the following:

The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.

When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.

The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.

For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

See Standards in the in FHWA HEC No. 11 for details on riprap materials and use of filter fabric.

Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in Section 4.4, *Open Channel Design.*

4.7.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.

Example 1

Given:	Box culvert - 8 ft by 6 ft	Design Discharge Q = 800 cfs
	Supercritical flow in culvert	Normal flow depth = brink depth
	Y_o = 4 ft	Tailwater depth TW = 2.8 ft

Find: Riprap basin dimensions for these conditions

Solution: Definition of terms in Steps 1 through 5 can be found in Figures 4.7-5 and 4.7-8.

$y_o = y_e$ for rectangular section; therefore, with y_o given as 4 ft, y_e = 4 ft.

$V_o = Q/A = 800/(4 \times 8) = 25$ ft/s

Froude Number = $Fr = V/(g \times y_e)^{0.5}$ ($g = 32.2$ ft/s^2)

$Fr = 25/(32.2 \times 4)^{0.5} = 2.20 < 2.5$ O.K.

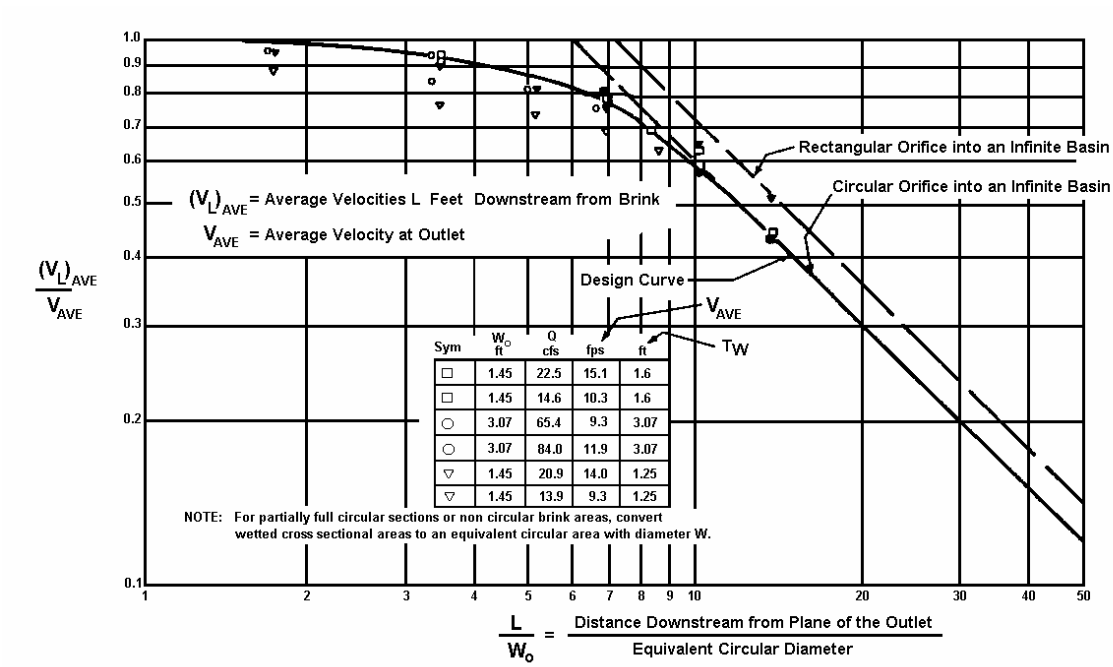


Figure 4.7-9 Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails
(Source: USDOT, FHWA, HEC-14, 1983)

$TW/y_e = 2.8/4.0 = 0.7$ Therefore, $TW/y_e < 0.75$ OK

Try $d_{50}/y_e = 0.45$, $d_{50} = 0.45 \times 4 = 1.80$ ft
From Figure 4.7-8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ ft
$h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 6.4 = 64$ ft (L_s = length of energy dissipator pool)
L_s min $= 3 \times W_o = 3 \times 8 = 24$ ft; therefore, use $L_s = 64$ ft

$L_B = 15 \times h_s = 15 \times 6.4 = 96$ ft (L_B = overall length of riprap basin)
L_B min $= 4 \times W_o = 4 \times 8 = 32$ ft; therefore, use $L_B = 96$ ft

Thickness of riprap: On the approach $= 3 \times d_{50} = 3 \times 1.8 = 5.4$ ft

Remainder $= 2 \times d_{50} = 2 \times 1.8 = 3.6$ ft
Other basin dimensions designed according to details shown in Figure 4.7-5.

Example 2

Given: Same design data as Example 1 except:

Tailwater depth TW = 4.2 ft
Downstream channel can tolerate only 7 ft/s discharge

Find: Riprap basin dimensions for these conditions

Solutions: Note -- High tailwater depth, $TW/y_o = 4.2/4 = 1.05 > 0.75$

From Example 1: $d_{50} = 1.8$ ft, $h_s = 6.4$ ft, $L_s = 64$ ft, $L_B = 96$ ft.

Design riprap for downstream channel. Use Figure 4.7-9 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32\ ft^2$

$D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft

$V_o = 25$ ft/s (From Example 1)

Set up the following table:

L/D_e	L (ft)	V_L/V_o	v_1 (ft/s)	Rock Size d_{50} (ft)
(Assume)	(Compute)	(Fig. 4.7-9)	(Fig. 4.7-1)	$D_e = W_o$
10	64	0.59	14.7	1.4
15*	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

*L/W_o is on a logarithmic scale so interpolations must be done logarithmically.

Riprap should be at least the size shown but can be larger. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication.

Example 3

Given: 6-ft diameter CMC
Design discharge Q = 135 cfs
Slope channel $S_o = 0.004$
Manning's n = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 ft
Normal velocity is 5.9 ft/s
Flow is subcritical
Tailwater depth TW = 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

Determine y_o and V_o

$Q/D^{2.5} = 135/6^{2.5} = 1.53$

TW/D = 2.0/6 = 0.33

From Figure 4.7-7, $y_o/D = 0.45$

$y_o = .45 \times 6 = 2.7$ ft

$TW/y_o = 2.0/2.7 = 0.74$ $TW/y_o < 0.75$ O.K.

Determine Brink Area (A) for $y_o/D = 0.45$

From Uniform Flow in Circular Sections Table (from Section 4.3)

For $y_o/D = d/D = 0.45$

$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3\ ft^2$

$V_o = Q/A = 135/12.3 = 11.0$ ft/s

For Froude number calculations at brink conditions,
$y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

Froude number = $Fr = V_o/(32.2 \times y_e)^{1/2} = 11/(32.2 \times 2.48)^{1/2} = 1.23 < 2.5$ OK

For most satisfactory results, $0.25 < d_{50}/y_e < 0.45$
Try $d_{50}/y_e = 0.25$
$d_{50} = 0.25 \times 2.48 = 0.62$ ft
From Figure 4.7-8, $h_s/y_e = 0.7$; therefore, $h_s = 0.7 \times 2.48 = 1.74$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From Section 4.3)
Check: $h_s/d_{50} = 1.74/0.62 = 2.8$, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 1.74 = 17.4$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft;
therefore, use $L_s = 17.4$ ft

$L_B = 15 \times h_s = 15 \times 1.74 = 26.1$ ft or $L_B = 4 \times W_o = 4 \times 6 = 24$ ft;
therefore, use $L_B = 26.1$ ft

$d_{50} = 0.62$ ft or use $d_{50} = 8$ in

Other basin dimensions should be designed in accordance with details shown on Figure 4.7-5. Figure 4.7-10 is provided as a convenient form to organize and present the results of riprap basin designs.

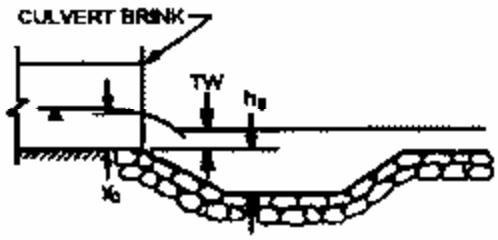
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x d_{50} thickness of riprap on the approach and the 2 x d_{50} thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

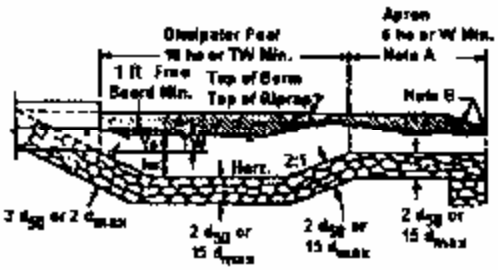
RIPRAP BASIN

Project No. _____

Designer _____ Date _____

Reviewer _____ Date _____





DESIGN VALUES	TRIAL 1	FINAL TRIAL
Equi. Depth, d_e		
D_{50}/d_e		
D_{50}		
Froude No., Fr		
h_s/d_e		
h_s		
h_s/D_{50}		
$2 < h_s/D_{50} < 4$		

BASIN DIMENSIONS		FEET
Pool length is the larger of:	$10h_s$	
	$3W_o$	
Basin length is the larger of:	$15h_s$	
	$4W_o$	
Approach Thickness	$3D_{50}$	
Basin Thickness	$2D_{50}$	

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_e	
TW/d_e	
IF $TW/d_e > 0.75$, calculate riprap downstream	
$D_e = (4A_o/\pi)^{0.5}$	

DOWNSTREAM RIPRAP				
L/D_e	L	V_L/V_o	V_L	D_{50}

Figure 4.7-10 Riprap Basin Design Form
(Source: USDOT, FHWA, HEC-14, 1983)

4.7.6 Baffled Outlets

4.7.6.1 Description

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 4.7-11. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

4.7.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 4.7-11 should be calculated as follows:

Determine input parameters, including:

- h = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)
- Q = Design discharge (cfs)
- v = Theoretical velocity (ft/s = 2gh)
- A = Q/v = Flow area (ft^2)
- d = $A^{0.5}$= Representative flow depth entering the basin (ft) *assumes square jet*
- Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless
- g = Acceleration of gravity (32.2 ft/s)

Calculate the minimum basin width, W, in ft, using the following equation.

$$W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566} \qquad (4.7.2)$$

Where:

- W = minimum basin width (ft)
- d = depth of incoming flow (ft)
- Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

Calculate the other basin dimensions as shown in Figure 4.7-11, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

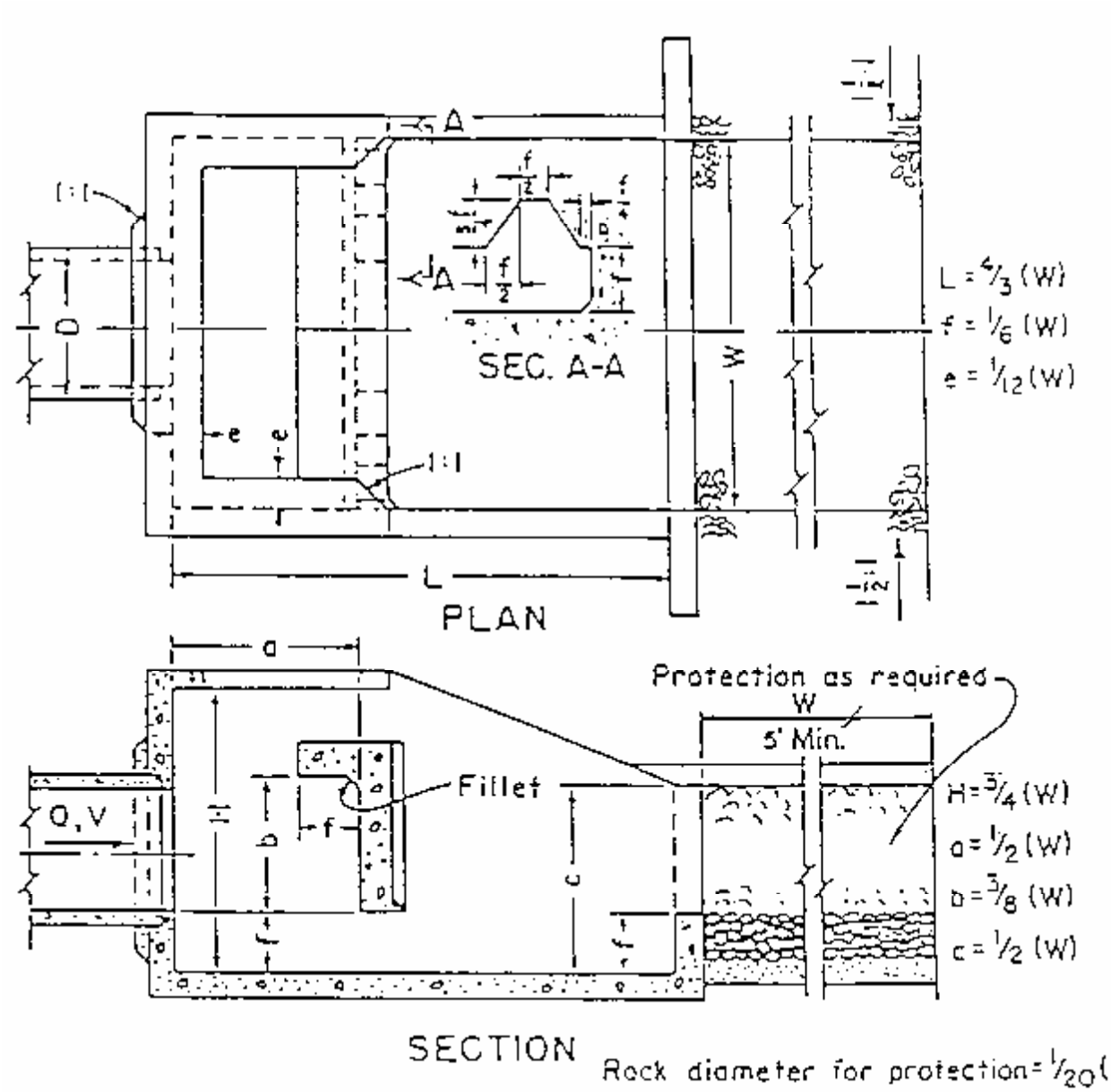


Figure 4.7-11 Schematic of Baffled Outlet
(Source: U.S. Dept. of the Interior, 1978)

If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

4.7.6.3 Example Design

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from
   $v = (2gh)^{0.5} = [2(32.2\ ft/sec^2)(15\ ft)]^{0.5} = 31.1\ ft/s$
   This is less than 50 ft/s, so a baffled outlet is suitable.

2. Determine the flow area using the theoretical velocity as follows:
   $A = Q/v = 150\ cfs/31.1\ ft/sec = 4.8\ ft^2$

3. Compute the flow depth using the area from Step 2.
   $d = (A)^{0.5} = (4.8\ ft^2)^{0.5} = 2.12\ ft$

4. Compute the Froude number using the results from Steps 1 and 3.
   $Fr = v/(gd)^{0.5} = 31.1\ ft/sec/[(32.2\ ft/sec^2)(2.12\ ft)]^{0.5} = 3.8$

5. Determine the basin width using equation 4.7.2 with the Froude number from Step 4.
   $W = 2.88\ dFr^{0.566} = 2.88\ (2.12)\ (3.8)^{0.566} = 13.0\ ft$ (minimum)
   Use 13 ft as the design width.

6. Compute the remaining basin dimensions (as shown in Figure 4.7-11):
   L = 4/3 (W) = 17.3 ft, use L = 17 ft, 4 in
   f = 1/6 (W) = 2.17 ft, use f = 2 ft, 2 in
   e = 1/12 (W) = 1.08 ft, use e = 1 ft, 1 in
   H = 3/4 (W) = 9.75 ft, use H = 9 ft, 9 in
   a = 1/2 (W) = 6.5 ft, use a = 6 ft, 6 in
   b = 3/8 (W) = 4.88 ft, use b = 4 ft, 11 in
   c = 1/2 (W) = 6.5 ft, use c = 6 ft, 6 in

   Baffle opening dimensions would be calculated as shown in Figure 4.7-11.

7. Basin invert should be at b/2 + f below tailwater, or
   (4 ft, 11 in)/2 + 2 ft, 2 in = 4.73 ft
   Use 4 ft 8 in; therefore, invert should be 2 ft, 8 in below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20, or about 8 in.

9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/s.
   $(3.14d)^2/4 = Q/v;\ d = [(4Q)/3.14v)]^{0.5} = [(4(150\ cfs)/3.14(12\ ft/sec)]^{0.5} = 3.99\ ft$
   Use 48-in pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 in.

4.7.7 Grade Control Structures

When channels are relocated through non-stable soils and stream gradients are increased, the stream bottom may degrade or dig itself deeper. This can cause bank instability, increased upstream scouring, and sloughing of natural slopes. The U.S. Soil Conservation Services (SCS) requires that streambed stability be maintained in any of its stream projects. This can be accomplished by grade stabilization structures; in essence a series of low-head weirs.

If designed and constructed with ecological values in mind, these structures can double as habitat enhancement devices. If improperly planned however, they can atually degrade habitat values. The most productive method of installing these structures is to use low weirs that pool water just a short distance (approximately 100 feet) upstream. A plunge pool will form just below the structures, and a riffle area should develop below this pool. The next structure should be located downstream a sufficient distance to avoid impounding the riffle area below the pool at the base of the upstream weir.

Specific construction requirements and techniques can be obtained from the SCS or other agencies upon request. The intent of this general discussion of grade stabilization structures is to promote consideration of such measures early in the planning process.

Source: US Army Corp of Engineers, Nashville District, *"Mitigating the Impacts of Stream Alterations"*, unkn.

References

Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14.

Federal Highway Administration, 1967. Use of Riprap for Bank Protection. Hydraulic Engineering Circular No. 11.

Searcy, James K., 1967. Use of Riprap for Bank Protection. Federal Highway Administration.

USDA, SCS, 1975. Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas, College Park, Maryland.

U.S. Department of Interior, Bureau of Reclamation, 1978. Design of Small Canal Structures.

CHAPTER

5

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CHAPTER

5

STORM WATER CONTROLS

Section 5.1
Storm Water Controls Overview

5.1.1 Storm Water Controls - Categories and Applicability

5.1.1.1 Introduction

Structural storm water controls are engineered facilities intended to treat storm water runoff and/or mitigate the effects of increased storm water runoff peak rate, volume, and velocity due to urbanization. This section provides an overview of structural storm water controls that can be used to address the minimum storm water management standards outlined in Section 1.2.

In terms of the *Integrated* Design Approach, a structural storm water control, or set of structural controls, must:

- **Water Quality:** Remove pollutants in storm water runoff to protect water quality;
- **Streambank Protection:** Regulate discharge from the site to minimize downstream bank and channel erosion; and
- **Flood Control:** Control conveyance of runoff within and from the site to minimize flood risk to people and properties.

5.1.1.2 Control Categories

The storm water control practices recommended in this Manual vary in their applicability and ability to meet storm water management goals:

Primary Controls

Primary Structural Storm Water Controls have the ability to fully address one or more of the Steps in the *integrated* Design Approach if designed appropriately. Structural controls are recommended for use with a wide variety of land uses and development types. These structural controls have a demonstrated ability to effectively treat the Water Quality Volume (WQ_v) and have been shown to be able to remove 70% to 80% of the annual average total suspended solids (TSS) load in typical post-development urban runoff when designed, constructed, and maintained in accordance with recommended specifications. Several of these structural controls can also be designed to provide primary control for downstream streambank protection (SP_v) and flood control (Q_f). These structural controls are recommended storm water management facilities for a site wherever feasible and practical.

Secondary Controls

However, a number of structural controls are recommended <u>only</u> for limited use or for special site or design conditions. Generally, these practices either: (1) do not have the ability on their own to fully address one or more of the Steps in the *integrated* Design Approach, (2) are intended to address hotspot

or specific land use constraints or conditions, and/or (3) may have high or special maintenance requirements that may preclude their use. These types of structural controls are typically used for *water quality treatment only*. Some of these controls can be used as a pretreatment measure or in series with other structural controls to meet pollutant removal goals. Such structural controls should be considered mostly for commercial, industrial, or institutional developments.

Table 5.1.1-1 lists the structural storm water control practices. These structural controls are recommended for use in a wide variety of applications. A detailed discussion of each of the controls, as well as design criteria and procedures can be found in Section 5.2.

Table 5.1.1-1 Structural Controls

Structural Control	Description
Bioretention Areas	*Bioretention areas* are shallow storm water basins or landscaped areas which utilize engineered soils and vegetation to capture and treat storm water runoff. Runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil.
Channels • Enhanced Swale (Dry, Wet, or Wetland) • Grass Channel (biofilter)	• *Enhanced swales* are vegetated open channels that are explicitly designed and constructed to capture and treat storm water runoff within dry or wet cells formed by check dams or other means • *Grass channels* provide "biofiltering" of storm water runoff as it flows across the grass surface. However, a grass channel alone cannot meet the 70% TSS removal performance goal. Consequently, grass channels should only be used as pretreatment measure or as part of a treatment train approach.
Chemical Treatment • Alum Treatment	*Alum treatment* provides for the removal of suspended solids from storm water runoff entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. Alum treatment should only be considered for large-scale projects where high water quality is desired.
Conveyance Components • Culvert • Inlet • Pipe Systems • Energy Dissipators • Open Conveyance Channel	• A *culvert* is a short, closed (covered) conduit that conveys storm water runoff under an embankment, usually a roadway. • *Inlets* are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. • *Pipe systems* are used for transporting runoff from roadway and other inlets to outfalls at structural storm water controls and receiving waters. Culverts, inlets, and pipe systems alone do not provide water quality treatment.
Detention • Dry Detention / Dry Extended Detention Basins • Multi-Purpose Detention Areas • Underground Detention	• *Dry detention basins and dry extended detention (ED) basins* are surface facilities intended to provide for the temporary storage of storm water runoff to reduce downstream water quantity impacts. • *Multi-purpose detention areas* are site areas used for one or more specific activities, such as parking lots and rooftops, which are also designed for the temporary storage of runoff. • *Underground detention tanks and vaults* are an alternative to surface dry detention for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area.

Table 5.1.1-1 Structural Controls

Structural Control	Description
Filtration • Filter Strip • Organic Filter • Planter Boxes • Surface Sand Filter/ Perimeter Sand Filter • Underground Sand Filter	• *Filter strips* provide "biofiltering" of storm water runoff as it flows across the grass surface. However, filter strips alone cannot meet the 70% TSS removal performance goal. Consequently, filter strips should only be used as pretreatment measure or as part of a treatment train approach. • *Organic filters* are surface sand filters where organic materials such as a leaf compost or peat/sand mixture are used as the filter media. These media may be able to provide enhanced removal of some contaminants, such as heavy metals. Given their potentially high maintenance requirements, they should only be used in environments that warrant their use. • *Planter boxes* are used on impervious surfaces in highly urbanized areas to collect and detain / infiltrate rainfall and runoff. The boxes may be prefabricated or constructed in place and contain growing medium, plants, and a reservoir. • *Sand filters* are multi-chamber structures designed to treat storm water runoff through filtration, using a sand bed as its primary filter media. Filtered runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil. • *Underground sand filters* are sand filter systems located in an underground vault. These systems should only be considered for extremely high density or space-limited sites.
Hydrodynamic Devices • Gravity (Oil-Grit) Separator	*Hydrodynamic controls* use the movement of storm water runoff through a specially designed structure to remove target pollutants. They are typically used on smaller impervious commercial sites and urban hotspots. These controls typically do not meet the Primary TSS removal performance goal and therefore should only be used as a pretreatment measure and as part of a treatment train approach.
Infiltration • Downspout Dry Wells • Infiltration Trench • Soakage Trenches	• *Downspout dry wells* are essentially perforated manholes, but they can be manufactured in various sizes. Located underground, they allow storm water infiltration even in highly urbanized areas. They should be used in conjunction with some type of pretreatment devices where there are minimal risks of groundwater contamination. • An *infiltration trench* is an excavated trench filled with stone aggregate used to capture and allow infiltration of storm water runoff into the surrounding soils from the bottom and sides of the trench. • *Soakage trenches* are a variation of infiltration trenches. Soakage trenches drain through a perforated pipe buried in gravel. They are used in highly impervious areas where conditions do not allow surface infiltration and where pollutant concentrations in runoff are minimal (i.e. non-industrial rooftops). They may be used in conjunction with other storm water devices, such as draining downspouts or planter boxes.

Table 5.1.1-1 Structural Controls

Structural Control	Description
Storm Water Ponds • Micropool Extended Detention Pond • Multiple Pond Systems • Wet Extended Detention Pond • Wet Pond	*Storm water ponds* are constructed storm water retention basins that have a permanent pool (or micropool) of water. Runoff from each rain event is detained and treated in the pool.
Porous Surfaces • Green Roofs • Modular Porous Paver Systems • Porous Concrete	• A *green roof* uses a small amount of substrate over an impermeable membrane to support a covering of plants. The green roof slows down runoff from the otherwise impervious roof surface as well as moderating rooftop temperatures. With the right plants, a green roof will also provide aesthetic or habitat benefits. • *Modular porous paver systems* consist of open void paver units laid on a gravel subgrade. Both porous concrete and porous paver systems provide water quality and quantity benefits, but have high workmanship and maintenance requirements, as well as high failure rates. • *Porous surfaces* are permeable pavement surfaces with an underlying stone reservoir to temporarily store surface runoff before it infiltrates into the subsoil. *Porous concrete* is the term for a mixture of course aggregate, Portland cement, and water that allows for rapid infiltration of water.
Proprietary Systems • Commercial Storm Water Controls	*Proprietary controls* are manufactured structural control systems available from commercial vendors designed to treat storm water runoff and/or provide water quantity control. Proprietary systems often can be used on small sites and in space-limited areas, as well as in pretreatment applications. However, proprietary systems are often more costly than other alternatives, may have high maintenance requirements, and often lack adequate independent performance data.
Re-Use • Rain Harvesting (tanks/barrels)	*Rain harvesting* is a container or system designed to capture and store rainwater discharged from a roof. The rain harvesting system consists of a storage container, a downspout diversion, a sealed lid, and an overflow system. Typical rain harvesting systems hold between 50 and 500 gallons of water and may work in series to provide larger volumes of storage.
Storm Water Wetlands • Extended Detention Shallow Wetland • Pocket Wetland • Pond/Wetland Systems • Shallow Wetland • Submerged Gravel Wetlands	• *Storm water wetlands* are constructed wetland systems used for storm water management. Storm water wetlands consist of a combination of shallow marsh areas, open water, and semi-wet areas above the permanent water surface. • *Submerged gravel wetland systems* use wetland plants in submerged gravel or crushed rock media to remove storm water pollutants. These systems should only be used in mid- to high-density environments where the use of other structural controls may be precluded. The long-term maintenance burden of these systems is uncertain.

5.1.1.3 Using Other or New Structural Storm Water Controls

Innovative technologies should be allowed and encouraged providing there is sufficient documentation as to their effectiveness and reliability. Communities can allow controls not included in this Manual at their discretion, but should not do so without independently derived information concerning performance, maintenance, application requirements, and limitations.

More specifically, new structural storm water control designs will not be accepted for inclusion in the manual until independent performance data shows that the structural control conforms to local and/or State criteria for treatment, conveyance, maintenance, and environmental impact.

5.1.2 Suitability of Storm Water Controls

Some structural storm water controls are intended to provide water quality treatment for storm water runoff. Though most of these structural controls provides pollutant removal capabilities, the relative capabilities vary between structural control practices and for different pollutant types.

5.1.2.1 Water Quality

Pollutant removal capabilities for a given structural storm water control practice are based on a number of factors including the physical, chemical, and/or biological processes that take place in the structural control and the design and sizing of the facility. In addition, pollutant removal efficiencies for the same structural control type and facility design can vary widely depending on the tributary land use and area, incoming pollutant concentration, flow rate, volume, pollutant loads, rainfall pattern, time of year, maintenance frequency, and numerous other factors.

To assist the designer in evaluating the relative pollutant removal performance of the various structural control options, Table 5.1.2-1 provides design removal efficiencies for each of the control practices. It should be noted that these values are *conservative* average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. A structural control design may be capable of exceeding these performances, however the values in the table are minimum reasonable values that can be assumed to be achieved when the structural control is sized, designed, constructed, and maintained in accordance with recommended specifications in this Manual.

Where the pollutant removal capabilities of an individual structural storm water control are not deemed sufficient for a given site application, additional controls may be used in series in a “treatment train” approach. More detail on using structural storm water controls in series is provided in subsection 5.1.6.

For additional information and data on the range of pollutant removal capabilities for various structural storm water controls, the reader is referred to the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Storm Water Best Management Practices (BMP) Database at www.bmpdatabase.org

Table 5.1.2-1 Design Pollutant Removal Efficiencies for Storm Water Controls (Percentage)

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metals
Bioretention Areas	80	60	50	---	80
Grass Channel	50	25	20	---	30
Enhanced Dry Swale	80	50	50	---	40
Enhanced Wet Swale	80	25	40	---	20
Alum Treatment	80	80	60	90	75

Table 5.1.2-1 Design Pollutant Removal Efficiencies for Storm Water Controls (Percentage)

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metals
Filter Strip	50	20	20	---	40
Dry Detention	65	50	30	70	---
Organic Filter	80	60	40	50	75
Planter Boxes	80	60	40	50	60
Sand Filters	80	50	25	40	50
Underground Sand Filter	80	50	25	40	50
Gravity (Oil-Grit) Separator	40	5	5	---	---
Downspout Drywell	80	60	60	90	90
Infiltration Trench	80	60	60	90	90
Soakage Trench	80	60	60	90	90
Storm Water Ponds	80	50	30	70*	50
Green Roof	85	---	25	---	95
Modular Porous Paver Systems with infiltration	**	80	80	---	90
Porous Concrete with infiltration	**	50	65	---	60
Proprietary Systems	***	***	***	***	***
Rain Harvesting	---	---	---	---	---
Storm Water Wetlands	80	40	30	70*	50
Submerged Gravel Wetland	80	50	20	70	50

* If no resident waterfowl population present

** Due to the potential for clogging, porous concrete and modular block paver systems should not be used for the removal of sediment or other coarse particulate pollutants

*** The performance of specific proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

--- Insufficient data to provide design removal efficiency

5.1.2.2 Streambank Protection

These controls have the ability to detain the volume and regulate the discharge of the 1-year, 24-hour storm event to protect natural waterways downstream of the development. Controls that provide streambank protection include detention, energy dissipation, storm water ponds, storm water wetlands, and pipe systems.

5.1.2.3 Flood Control

- **On-Site:** These controls have the ability to safely convey storm water through a development to minimize the flood risk to persons and property on-site. On-site flood control structures include channels, culverts, detentions, enhanced swales, open conveyance channels, storm water ponds, conveyance components (inlets and pipe systems), and storm water wetlands.
- **Downstream:** These controls have the ability to detain the volume and regulate the discharge from the controlling storm event, as determined by downstream assessment, and to minimize flood risk to persons and property downstream of the development. Downstream flood controls include open channels, pipe systems, detention, storm water ponds, and storm water wetlands.

5.1.3 Storm Water Control Selection

5.1.3.1 Control Screening Process

Outlined below is a screening process for structural storm water controls which can effectively treat the water quality volume as well as provide water quantity control. This process is intended to assist the site designer and design engineer in the selection of the most appropriate structural controls for a development site, and provides guidance on factors to consider in their location.

In general the following four criteria should be evaluated in order to select the appropriate structural control(s) or group of controls for a development:

- Storm Water Treatment Suitability
- Water Quality Performance
- Site Applicability
- Implementation Considerations

In addition, for a given site, the following factors should be considered and any specific design criteria or restrictions need to be evaluated:

- Physiographic Factors
- Soils
- Special Watershed or Stream Considerations

Finally, environmental regulations should be considered as they may influence the location of a structural control on site, or may require a permit.

The following pages provide a selection process for comparing and evaluating various structural storm water controls using a screening matrix and a list of location and permitting factors. These tools are provided to assist the design engineer in selecting the subset of structural controls that will meet the storm water management and design objectives for a development site or project.

Step 1 Overall Applicability

Through the use of the first four screening categories in Table 5.1.3-1, the site designer evaluates and screens the overall applicability of the full set of structural controls as well as the constraints of the site in question. The following are the details of the various screening categories and individual characteristics used to evaluate the structural controls.

<u>Storm Water Management Suitability</u>

The first category in the Matrix examines the capability of each structural control option to provide water quality treatment, downstream streambank protection, and flood control. A blank entry means that the structural control cannot or is not typically used to meet an *integrated* Design Approach. This does not necessarily mean that it should be eliminated from consideration, but rather is a reminder that more than one structural control may be needed at a site (e.g., a bioretention area used in conjunction with dry detention storage).

Ability to treat the Water Quality Volume (WQ_v). This indicates whether a structural control provides treatment of the water quality volume (WQ_v). The presence of a “P” or an “S” indicates whether the control is a Primary or Secondary control for meeting the TSS reduction goal.

Ability to provide Streambank Protection (SP_v). This indicates whether the structural control can be used to provide the extended detention of the streambank protection volume (SP_v). The presence of a “P” indicates that the structural control can be used to meet SP_v requirements. An “S” indicates that

the structural control may be sized to provide streambank protection in certain situations, for instance on small sites.

Ability to provide Flood Control (Q_f). This indicates whether a structural control can be used to meet the flood control criteria. The presence of a "P" indicates that the structural control can be used to provide peak reduction of the 100-year storm event.

Relative Water Quality Performance

The second category of the Matrix provides an overview of the pollutant removal performance of each structural control option, when designed, constructed, and maintained according to the criteria and specifications in this Manual.

Ability to provide TSS and Sediment Removal. This column indicates the capability of a structural control to remove sediment in runoff. All of the Primary structural controls are presumed to remove 70% to 80% of the average annual total suspended solids (TSS) load in typical urban post-development runoff (and a proportional removal of other pollutants).

Ability to provide Nutrient Treatment. This column indicates the capability of a structural control to remove the nutrients nitrogen and phosphorus in runoff, which may be of particular concern with certain downstream receiving waters.

Ability to provide Bacteria Removal. This column indicates the capability of a structural control to remove bacteria in runoff. This capability may be of particular focus in areas with public beaches, shellfish beds, or to meet water regulatory quality criteria under the Total Maximum Daily Load (TMDL) program.

Ability to accept Hotspot Runoff. This last column indicates the capability of a structural control to treat runoff from designated hotspots. Hotspots are land uses or activities which produce higher concentrations of trace metals, hydrocarbons, or other priority pollutants. Examples of hotspots might include: gas stations, convenience stores, marinas, public works storage areas, garbage transfer facilities, material storage sites, vehicle service and maintenance areas, commercial nurseries, vehicle washing/steam cleaning, landfills, construction sites, industrial sites, industrial rooftops, and auto salvage or recycling facilities. A check mark indicates that the structural control may be used on hotspot site; however, it may have specific design restrictions. Please see the specific design criteria of the structural control for more details. Local jurisdictions may have other site uses which they designate as Hotspots, so their criteria should be checked as well.

Site Applicability

The third category of the Matrix provides an overview of the specific site conditions or criteria that must be met for a particular structural control to be suitable. In some cases, these values are recommended values or limits and can be exceeded or reduced with proper design or depending on specific circumstances. Please see the specific criteria section of the structural control for more details.

Drainage Area. This column indicates the approximate minimum or maximum drainage area considered suitable for the structural control practice. If the drainage area present at a site is slightly greater than the maximum allowable drainage area for a practice, some leeway can be permitted if more than one practice can be installed. The minimum drainage areas indicated for ponds and wetlands should not be considered inflexible limits, and may be increased or decreased depending on water availability (baseflow or groundwater), the mechanisms employed to prevent outlet clogging, or design variations used to maintain a permanent pool (e.g., liners).

Space Required (Space Consumed). This comparative index expresses how much space a structural control typically consumes at a site in terms of the approximate area required as a percentage of the impervious area draining to the control.

Slope. This column evaluates the effect of slope on the structural control practice. Specifically, the slope restrictions refer to how flat the area where the facility is installed must be and/or how steep the contributing drainage area or flow length can be.

Minimum Head. This column provides an estimate of the minimum elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the structural control.

Water Table. This column indicates the minimum depth to the seasonally high water table from the bottom or floor of a structural control.

Implementation Considerations

The fourth category in the Matrix provides additional considerations for the applicability of each structural control option.

Residential Subdivision Use. This column identifies whether or not a structural control is suitable for typical residential subdivision development (not including high-density or ultra-urban areas).

Ultra-Urban. This column identifies those structural controls appropriate for use in very high-density (ultra-urban) areas, or areas where space is a premium.

Construction Cost. The structural controls are ranked according to their relative construction cost per impervious acre treated, as determined from cost surveys.

Maintenance. This column assesses the relative maintenance effort needed for a structural storm water control, in terms of three criteria: frequency of scheduled maintenance, chronic maintenance problems (such as clogging), and reported failure rates. It should be noted that **all structural controls** require routine inspection and maintenance.

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Table 5.1.3-1 Structural Control Screening Matrix

Category	On-Site Storm Water Controls	STORM WATER TREATMENT SUITABILITY				WATER QUALITY PERFORMANCE				SITE APPLICABILITY					IMPLEMENTATION CONSIDERATIONS			
		Water Quality Protection	Streambank Protection	On-Site Flood Control	Downstream Flood Control	TSS/ Sediment Removal Rate	Nutrient Removal Rate (TP/TN)	Bacteria Removal Rate	Hotspot Application	Drainage Area (acres)	Space Req'd (% of tributary Imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table	Residential Subdivision Use	High Density/Ultra Urban	Capital Cost	Maintenance Burden
Bioretention Areas	Bioretention Areas	P	S	S	-	80%	60%/50%	-	✓	5 max***	5-7%	6% max	5 ft	2 feet	✓	✓	Moderate	Low
Channels	Enhanced Swales	P	S	S	S	80%	25%/40%	-	✓	5 max	10-20%	4% max	1 ft	below WT	✓		High	Low
	Channels, Grass	S	S	P	S	50%	25%/20%	-							✓		Low	Moderate
	Channels, Open	-	-	P	S	-	-	-							✓		Low	Low
Chemical Treatment	Alum Treatment System	P	-	-	-	90%	80%/60%	90%	✓	25 min	None				✓	✓	High	High
Conveyance Components	Culverts	-	-	P	P	-	-	-							✓	✓	Low	Low
	Energy Dissipation	-	P	S	S	-	-	-							✓	✓	Low	Low
	Inlets/Street Gutters	-	-	P	-	-	-	-							✓	✓	Low	Low
	Pipe Systems	-	P	P	P	-	-	-							✓	✓	Low	Low
Detention	Detention, Dry	S	P	P	P	65%	50%/30%	70%	✓		2 - 3%	15% across pond	6 to 8 ft	2 feet	✓		Low	Moderate to High
	Detention, Extended Dry	S	P	P	P	65%	50%/30%	70%	✓		2 - 3%	15% across pond	6 to 8 ft	2 feet	✓		Low	Moderate to High
	Detention, Multi-purpose Areas	-	P	P	P	-	-	-		200 max		1% for Parking Lot; 0.25 in/ft for Rooftop			✓	✓	Low	Low
	Detention, Underground	-	P	P	P	-	-	-		200 max						✓	High	Moderate
Filtration	Filter Strips	S	-	-	-	50%	20%/20%	-		2 max***	20-25%	2-6%			✓		Low	Moderate
	Organic Filters	P	-	-	-	80%	60%/40%	50%	✓	10 max***	2-3%		5 to 8 ft			✓	High	High
	Planter Boxes	P	-	-	-	80%	60%/40%	-			6%					✓	Low	Moderate
	Sand Filters, Surface/ Perimeter	P	S	-	-	80%	50%/25%	40%	✓	10 max***/ 2 max***	2-3%	6% max	5 ft/ 2 to 3 ft	2 feet		✓	High	High
	Sand Filters, Underground	P	-	-	-	80%	50%/25%	40%	✓	5 max	None					✓	High	High
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	S	-	-	-	40%	5%/5%	-		1 max***	None					✓	High	High
Infiltration	Downspout Drywell	P	-	-	-	80%	60%/60%	90%							✓	✓	Low	Moderate
	Infiltration Trenches	P	S	-	-	80%	60%/60%	90%		5 max	2-3%	6% max	1 ft	4 feet	✓	✓	High	High
	Soakage Trenches	P	S	-	-	80%	60%/60%	90%		5 max	27' per 1000 ft^2 impervious area	6% max	1 ft	4 feet	✓	✓	High	High
Ponds	Wet Pond	P	P	P	P	80%	50%/30%	70%*	✓		2-3%	15% max	6 to 8 ft	2 feet, if hotspot or aquifer	✓		Low	Low
	Wet ED Pond	P	P	P	P	80%	50%/30%	70%*	✓	25 min**					✓		Low	Low
	Micropool ED Pond	P	P	P	P	80%	50%/30%	70%*	✓	10 min**					✓		Low	Moderate
	Multiple ponds	P	P	P	P	80%	50%/30%	70%*	✓	25 min**					✓		Low	Low
Porous Surfaces	Green Roof	P	S	-	-	85%	95%/16%	-	✓							✓	High	High
	Modular Porous Paver Systems	S	S	-	-	**	80%/80%	-		5 max	Varies					✓	Moderate	High
	Porous Concrete	S	S	-	-	**	50%/65%	-		5 max	Varies					✓	High	High
Proprietary Systems	Proprietary Systems ****	S	S	S	S	****	****	****		****	****				****	✓	High	High
Re-Use	Rain Harvesting	P	-	-	-	-	-	-							✓	✓	Low	High
Wetlands	Wetlands, Storm Water	P	P	P	P	80%	40%/30%	70%*	✓	25 min	3-5%	8% max	3 to 5 ft (shallow) 6 to 8 ft (pond)	2 feet, if hotspot or aquifer	✓		Moderate	Moderate
	Wetlands, Submerged Gravel	P	P	S	-	80%	50%/20%	70%	✓	5 min			2 to 3 ft	below WT	✓	✓	Moderate	High

✓ - Meets suitability criteria

P - Primary Control, meets suitability criteria

S - Secondary Control, can be incorporated into the structural control in certain situations

* Provides less than 80% TSS removal efficiency. May be used in pretreatment and as part of a "treatment train"

** Smaller area acceptable with adequate water balance and anti-clogging device

*** Drainage area can be larger in some instances

**** The application and performance of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

1 Porous surfaces provide water quantity benefits by reducing the effective impervious area

2 Due to the potential for clogging, porous surfaces should not be used for the removal of sediment or other coarse particulate pollutants

Table 5.1.3-1 Structural Control Screening Matrix

Category	On-Site Storm Water Controls	PHYSIOGRAPHIC FACTORS			Soils	SPECIAL WATERSHED CONSIDERATIONS		
		Low Relief	High Relief	Karst		High Quality Stream	Aquifer Protection	Reservior Protection
Bioretention Areas	Bioretention Areas	Several design variations will likely be limited by low head		Use poly-liner or impermeable membrane to seal bottom	Clay or silty soils may require pretreatment	Evaluate for stream warming	Needs to be designed with no exfiltration (i.e. outflow to groundwater)	
Channels	Enhanced Swales	Generally feasible however slope <1% may lead to standing water in dry swales	Often infeasible if slopes are 4% or greater				Hotspot runoff must be adequately treated	Hotspot runoff must be adequately treated
	Channels, Grass							
	Channels, Open							
Chemical Treatment	Alum Treatment System							
Conveyance Components	Culverts							
	Energy Dissipation							
	Inlets/Street Gutters							
	Pipe Systems							
Detention	Detention, Dry		Embankment heights restricted	Require poly or clay liner, Max ponding depth, Geotechnical tests	Underlying soils of hydrologic group "C" or "D" should be adequate to maintain a permanent pool. Most group "A" soils and some group "B" soils will require a pond liner.			
	Detention, Extended Dry							
	Detention, Multi-purpose Areas							
	Detention, Underground			GENERALLY NOT ALLOWED				
Filtration	Filter Strips							
	Organic Filters							
	Planter Boxes				Type A or B			
	Sand Filters, Surface/ Perimeter	Several design variations will likely be limited by low head		Use poly-liner or impermeable membrane to seal bottom	Clay or silty soils may require pretreatment	Evaluate for stream warming	Needs to be designed with no exfiltration (i.e. outflow to groundwater)	
	Sand Filters, Underground							
Hydrodynamic Devices	Gravity (Oil-Grit) Separator							
Infiltration	Downspout Drywell	Minimum distance to water table of 4 feet		GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr			
	Infiltration Trenches	Minimum distance to water table of 2 feet	Maximum slope of 6% Trenches must have flat bottom	GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr		Maintain safe distance from wells and water table. No hotspot runoff	Maintain safe distance from bedrock and water table. Pretreat runoff
	Soakage Trenches	Minimum distance to water table of 4 feet	Maximum slope of 6% Trenches must have flat bottom	GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr			
Ponds	Wet Pond	Limit maximum normal pool depth to about 4 feet (dugout) Providing pond drain can be problematic	Embankment heights restricted	Require poly or clay liner Max ponding depth Geotechnical tests	"A" soils may require pond liner "B" soils may require infiltration testing	Evaluate for stream warming	May require liner if "A" soils are present Pretreat hotspots 2 to 4 ft separation distance from water table	
	Wet ED Pond							
	Micropool ED Pond							
	Multiple ponds							
Porous Surfaces	Green Roof							
	Modular Porous Paver Systems							
	Porous Concrete							
Proprietary Systems	Proprietary Systems *							
Re-Use	Rain Harvesting							
Wetlands	Wetlands, Storm Water		Embankment heights restricted	Require poly-liner Geotechnical tests	"A" soils may require pond liner	Evaluate for stream warming	May require liner if "A" soils are present Pretreat hotspots 2 to 4 ft separation distance from water table	
	Wetlands, Submerged Gravel							

✓ - Meets suitability criteria

P - Primary Control, meets suitability criteria

S - Secondary Control, can be incorporated into the structural control in certain situations

* Provides less than 80% TSS removal efficiency. May be used in pretreatment and as part of a "treatment train"

** Smaller area acceptable with adequate water balance and anti-clogging device

*** Drainage area can be larger in some instances

**** The application and performance of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

1 Porous surfaces provide water quantity benefits by reducing the effective impervious area

2 Due to the potential for clogging, porous surfaces should not be used for the removal of sediment or other coarse particulate pollutants

Step 2 Specific Criteria

The last three categories in the Structural Control Screening matrix (Table 5.1.3-1) provides an overview of various specific design criteria and specifications, or exclusions for a structural control that may be present due to a site's general physiographic character, soils, or location in a watershed with special water resources considerations.

Physiographic Factors

Three key factors to consider are low-relief, high-relief, and karst terrain. In the North Central Texas, low relief (very flat) areas are primarily located east of the Dallas metropolitan area. High relief (steep and hilly) areas are primarily located west of the Fort Worth metropolitan area. Karst and major carbonaceous rock areas are limited to portions of Palo Pinto, Erath, Hood, Johnson, and Somerveil counties. Special geotechnical testing requirements may be needed in karst areas. The local reviewing authority should be consulted to determine if a project is subject to terrain constraints.

- Low relief areas need special consideration because many structural controls require a hydraulic head to move storm water runoff through the facility.
- High relief may limit the use of some structural controls that need flat or gently sloping areas to settle out sediment or to reduce velocities. In other cases, high relief may impact dam heights to the point that a structural control becomes infeasible.
- Karst terrain can limit the use of some structural controls as the infiltration of polluted waters directly into underground streams found in karst areas may be prohibited. In addition, ponding areas may not reliably hold water in karst areas.

Soils

The key evaluation factors are based on an initial investigation of the NRCS hydrologic soils groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors.

Special Watershed or Stream Considerations

The design of structural storm water controls is fundamentally influenced by the nature of the downstream water body that will be receiving the storm water discharge. In addition, the designer should consult with the appropriate review authority to determine if their development project is subject to additional structural control criteria as a result of an adopted local watershed plan or special provision.

In some cases, higher pollutant removal or environmental performance is needed to fully protect aquatic resources and/or human health and safety within a particular watershed or receiving water. Therefore, special design criteria for a particular structural control or the exclusion of one or more controls may need to be considered within these watersheds or areas. Examples of important watershed factors to consider include:

High Quality Streams (Streams with a watershed impervious cover less than approximately 15%). These streams may also possess high quality cool water or warm water aquatic resources or endangered species. The design objectives are to maintain habitat quality through the same techniques used for cold-water streams, with the exception that stream warming is not as severe of a design constraint. These streams may also be specially designated by local authorities.

Wellhead Protection. Areas that recharge existing public water supply wells present a unique management challenge. The key design constraint is to prevent possible groundwater contamination by preventing infiltration of hotspot runoff. At the same time, recharge of unpolluted storm water is encouraged to maintain flow in streams and wells during dry weather.

Reservoir or Drinking Water Protection. Watersheds that deliver surface runoff to a public water supply reservoir or impoundment are a special concern. Depending on the treatment available, it may be necessary to achieve a greater level of pollutant removal for the pollutants of concern, such

as bacteria pathogens, nutrients, sediment, or metals. One particular management concern for reservoirs is ensuring storm water hotspots are adequately treated so they do not contaminate drinking water.

Step 3 Location and Permitting Considerations

In the last step, a site designer assesses the physical and environmental features at the site to determine the optimal location for the selected structural control or group of controls. The checklist below (Table 5.1.3-2) provides a condensed summary of current restrictions as they relate to common site features that may be regulated under local, state, or federal law. These restrictions fall into one of three general categories:

- Locating a structural control within an area when expressly prohibited by law.
- Locating a structural control within an area that is strongly discouraged, and is only allowed on a case by case basis. Local, state, and/or federal permits shall be obtained, and the applicant will need to supply additional documentation to justify locating the storm water control within the regulated area.
- Structural storm water controls must be setback a fixed distance from a site feature.

This checklist is only intended as a general guide to location and permitting requirements as they relate to siting of storm water structural controls. Consultation with the appropriate regulatory agency is the best strategy.

Table 5.1.3-2 Location and Permitting Checklist

Site Feature	Location and Permitting Guidance
Jurisdictional Wetland (Waters of the U.S) U.S. Army Corps of Engineers Regulattory Permit	• Jurisdictional wetlands should be delineated prior to siting structural control. • Use of natural wetlands for storm water quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Storm water should be treated prior to discharge into a natural wetland. • Structural controls may also be *restricted* in local buffer zones. Buffer zones may be utilized as a non-structural filter strip (i.e., accept sheet flow). • Should justify that no practical upland treatment alternatives exist. • Where practical, excess storm water flows should be conveyed away from jurisdictional wetlands.
Stream Channel (Waters of the U.S) U.S. Army Corps of Engineers Section 404 Permit	• All Waters of the U.S. (streams, ponds, lakes, etc.) should be delineated prior to design. • Use of any Waters of the U.S. for storm water quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Storm water should be treated prior to discharge into Waters of the U.S. • In-stream ponds for storm water quality treatment are highly discouraged. • Must justify that no practical upland treatment alternatives exist. • Temporary runoff storage preferred over permanent pools. • Implement measures that reduce downstream warming.

Table 5.1.3-2 Location and Permitting Checklist

Site Feature	Location and Permitting Guidance
Texas Commission on Environmental Quality Groundwater Management Areas	• Conserve, preserve, protect, recharge, and prevent waste of groundwater resources through Groundwater Conservation Districts • Groundwater Conservation District pending for Middle Trinity. • Detailed mapping available from Texas Alliance of Groundwater Districts.
Texas Commission on Environmental Quality Surface Water Quality Standards	• Specific stream and reservoir buffer requirements. • May be imperviousness limitations • May be specific structural control requirements. • TCEQ provides water quality certification – in conjunction with 404 permit • Mitigation will be required for imparts to existing aquatic and terrestrial habitat.
100 Year Floodplain Local Storm water review Authority	• Grading and fill for structural control construction is generally discouraged within the 100 year floodplain, as delineated by FEMA flood insurance rate maps, FEMA flood boundary and floodway maps, or more stringent local floodplain maps. • Floodplain fill cannot raise the floodplain water surface elevation by more than limits set by the appropriate jurisdiction.
Stream Buffer Check with appropriate review authority whether stream buffers are required	• Consult local authority for storm water policy. • Structural controls are discouraged in the streamside zone (within 25 feet or more of streambank, depending on the specific regulations).
Utilities Local Review Authority	• Call appropriate agency to locate existing utilities prior to design. • Note the location of proposed utilities to serve development. • Structural controls are discouraged within utility easements or rights of way for public or private utilities.
Roads TxDOT or DPW	• Consult TxDOT for any setback requirement from local roads. • Consult DOT for setbacks from State maintained roads. • Approval must also be obtained for any storm water discharges to a local or state-owned conveyance channel.
Structures Local Review Authority	• Consult local review authority for structural control setbacks from structures. • Recommended setbacks for each structural control group are provided in the performance criteria in this manual.
Septic Drain fields Local Health Authority	• Consult local health authority. • Recommended setback is a minimum of 50 feet from drain field edge or spray area.
Water Wells Local Health Authority	• 100-foot setback for storm water infiltration. • 50-foot setback for all other structural controls.

5.1.3.2 Example Application

A 20-acre institutional area (e.g., church and associated buildings) is being constructed in a dense urban area within Dallas/Fort Worth metropolitan area. The impervious coverage of the site is 40%. The site drains to an urban stream that is highly impacted from hydrologic alterations (accelerated channel erosion). The stream channel is deeply incised; consequently, flooding is not a problem. The channel drains to an urban river that is tributary to a phosphorus limited drinking water reservoir. Low permeability soils limit infiltration practices.

Objective: Avoid additional disruptions to receiving channel and reduce pollutant loads for sediment and phosphorus to receiving waters.

Target Removals: Provide storm water management to mitigate for accelerated channel incision and reduce loadings of key pollutants by the following:

- Sediment: 70% to 80%
- Phosphorus: 40%

Activity/Runoff Characteristics: The proposed site is to have large areas of impervious surface in the form of parking and structures. However, there will be a large contiguous portion of turf grass proposed for the front of the parcel that will have a relatively steep slope (approximately 10%) and will drain to the storm drain system associated with the entrance drive. Storm water runoff from the site is expected to exhibit fairly high sediment levels and seasonally high phosphorus levels (due to turf grass management).

Table 5.1.3-3 lists the results of the selection analysis using the screening matrix described previously. The highlighted rows indicate the controls selected for this example. The **X**'s indicate inadequacies in the control for this site. The ✓'s indicate adequate control capabilities for this site.

While there is a downstream reservoir to consider, there are no special watershed factors or physiographic factors to preclude the use of any of the practices from the structural control list. However, due to the size of the drainage area, most storm water ponds and wetlands are removed from consideration. In addition, the site's impermeable soils remove an infiltration trench from being considered. Due to the need to provide flood control as well as streambank protection storage, an extended detention micropool pond will likely be needed, unless some downstream regional storage is available to control flood waters.

To provide additional pollutant removal capabilities in an attempt to better meet the target removals, bioretention, surface sand filters, and/or perimeter sand filters can be used to treat the parking lot and driveway runoff. The bioretention will provide some removal of phosphorus while improving the aesthetics of the site. Surface sand filters provide higher phosphorus removal at a comparable unit cost to bioretention, but are not as aesthetically pleasing. The perimeter sand filter, is a flexible, easy to access practice (but at higher cost) that provides good phosphorus removal and additionally high oil and grease trapping ability.

The site drainage system can be designed so the bioretention and/or sand filters drain to the extended detention micropool pond for redundant treatment. Vegetated dry swales could also be used to convey runoff to the pond, which would provide pretreatment. Pocket wetlands and wet swales were eliminated from consideration due to potential for nuisance conditions. Underground sand filters could also be used at the site; however, cost and aesthetic considerations were significant enough to eliminate from consideration.

Table 5.1.3-3 Sample Structural Control Selection Matrix

Structural Control Alternative	Storm Water Treatment Suitability	Site Applicability	Implementation Considerations	Physiographic Factors/Soils	Special Watershed Considerations	Other Issues
Bioretention	✓[1]	✓[2]	✓	✓	none	
Dry Swale	✓[1]	✓[2]	✓	✓	none	
Wet Swale	✓[1]	✓[2]	✓	✓	none	Odor / mosquitoes
Perimeter Sand Filter	✓[1]	✓[2]	✓	✓	none	Higher cost
Surface Sand Filter	✓[1]	✓[2]	✓	✓	none	Aesthetics
Infiltration Trench	✓[1]	✓	✓	X		
Extended Detention Micropool Pond	✓	✓	✓	✓	none	
Multiple Ponds	✓	X				
Wet Extended Detention Pond	✓	X				
Wet Pond	✓	X				
Extended Detention Shallow Wetland	✓	X				
Pocket Wetland	✓	✓	✓	✓	none	Odor / mosquitoes
Shallow Wetland	✓	X				

Notes:
1. Only when used with another structural control that provides water quantity control
2. Can treat a portion of the site

5.1.4 On-Line Versus Off-Line Structural Controls

5.1.4.1 Introduction

Structural storm water controls are designed to be either "on-line" or "off-line." On-line facilities are designed to receive, but not necessarily control or treat, the entire runoff volume above the Q_f up to the peak 100-year discharge (Q_{p100}). On-line structural controls must be able to handle the entire range of storm flows.

Off-line facilities on the other hand are designed to receive only a specified flow rate or volume through the use of a flow regulator (i.e. diversion structure, flow splitter, etc). Flow regulators are typically used to divert the water quality volume (WQ_v) to an off-line structural control sized and designed to treat and control the WQ_v. After the design runoff flow has been treated and/or controlled, it is returned to the conveyance system. Figure 5.1.4.1 shows an example of an off-line sand filter and an on-line enhanced dry swale.

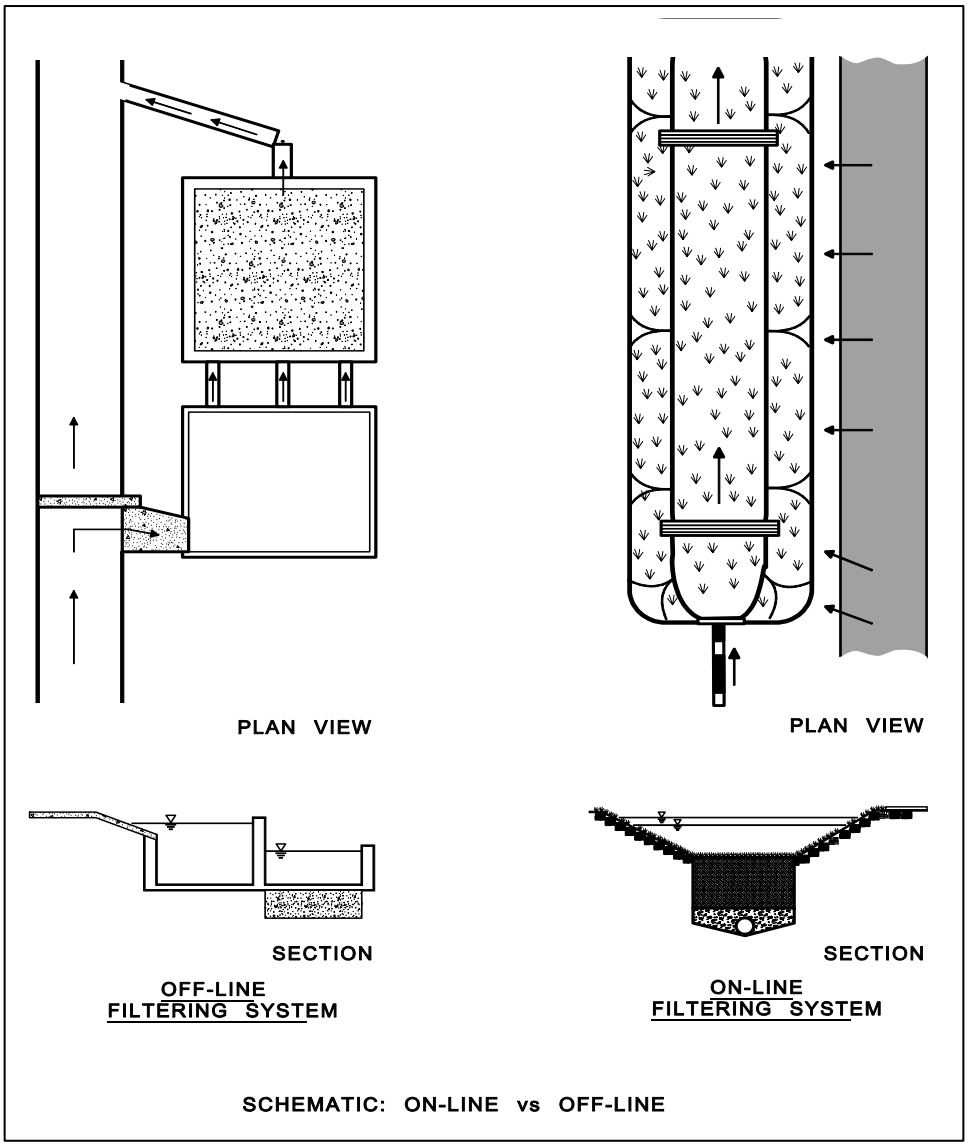


Figure 5.1.4-1 Example of On-Line versus Off-Line Structural Controls
(Source: CWP, 1996)

5.1.4.2 Flow Regulators

Flow regulation to off-line structural storm water controls can be achieved by either:

- Diverting the water quality volume or other specific maximum flow rate to an off-line structural storm water control, or
- Bypassing flows in excess of the design flow rate

The peak water quality flow rate (Q_{wq}) can be calculated using the procedure found in Section 2.1.10.2.

Flow regulators can be flow splitter devices, diversion structures, or overflow structures. A number of examples are shown below.

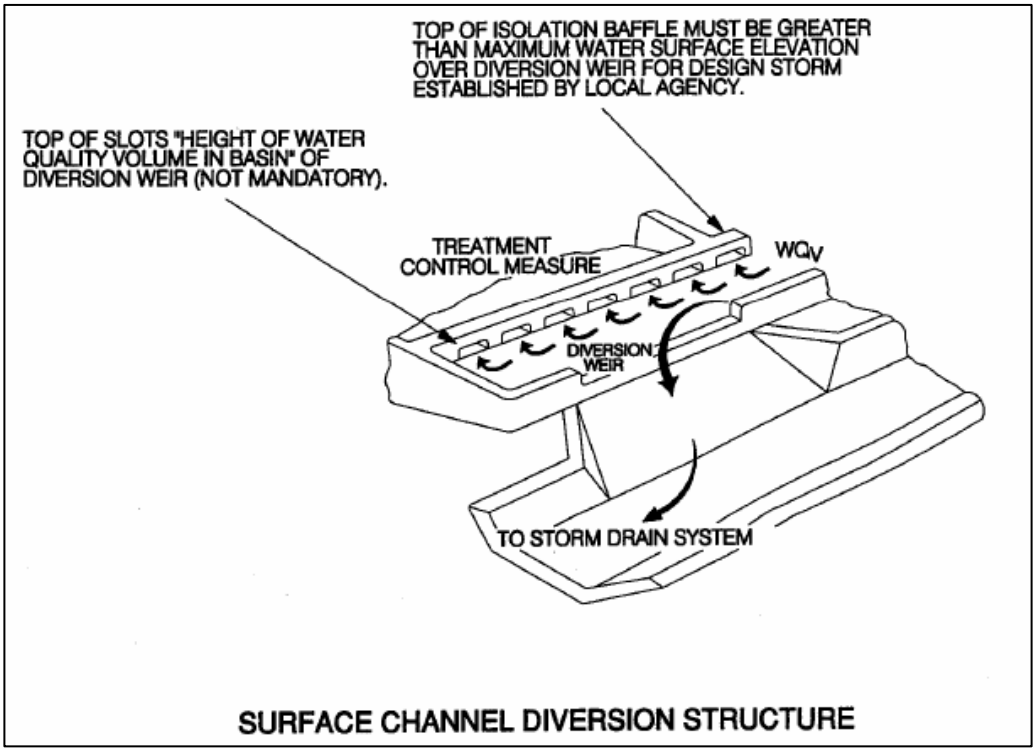


Figure 5.1.4-2 Pipe Interceptor Diversion Structure
(Source: City of Sacramento, 2000)

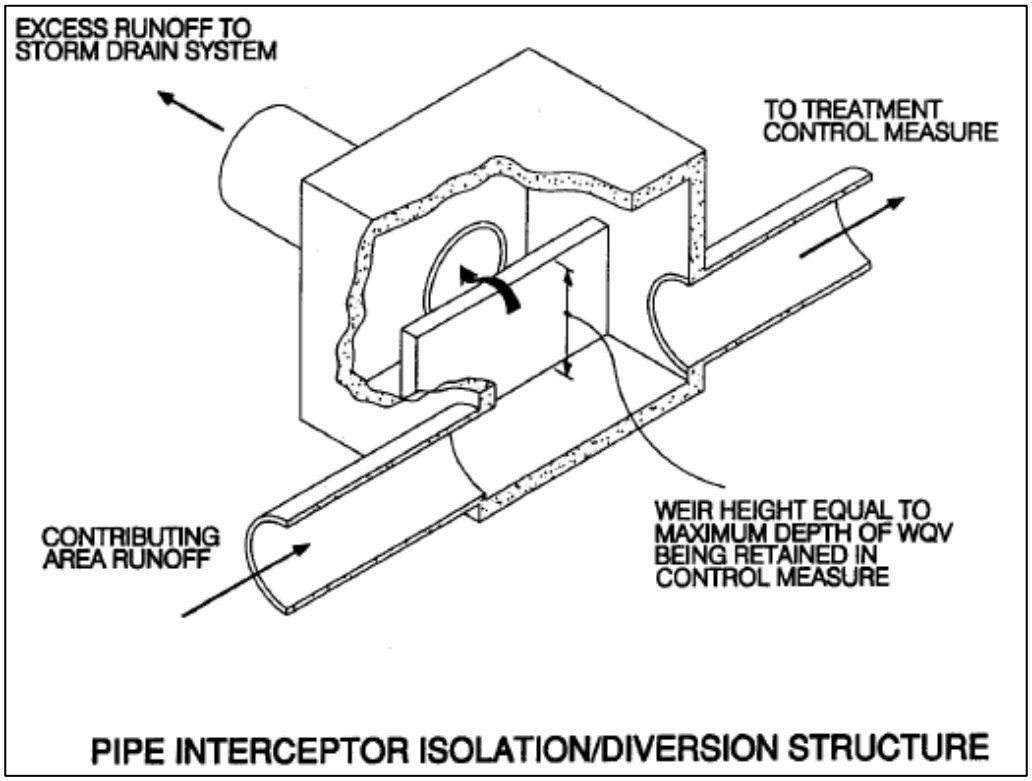


Figure 5.1.4-3 Surface Channel Diversion Structure
(Source: City of Sacramento, 2000)

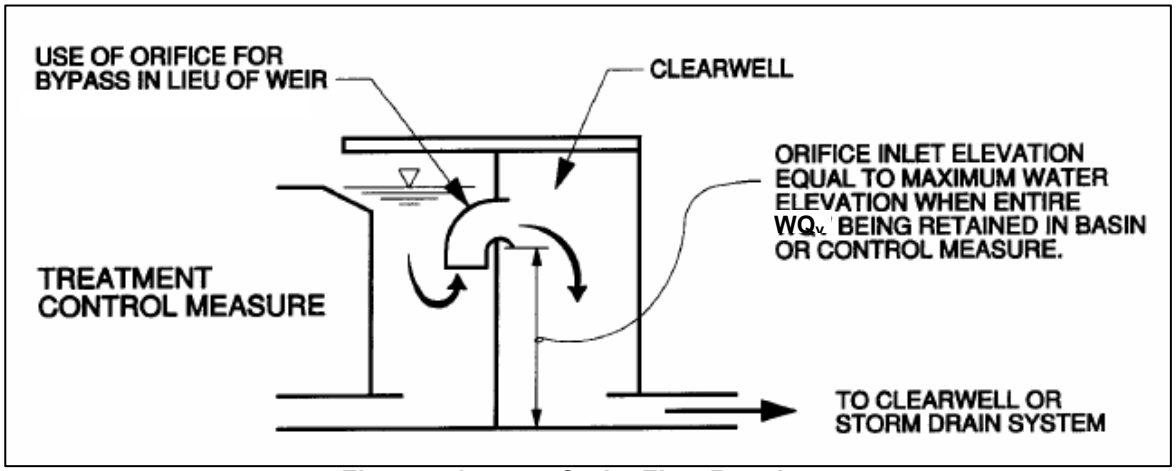


Figure 5.1.4-4 Outlet Flow Regulator
(Source: City of Sacramento, 2000)

5.1.5 Regional Versus On-Site Storm Water Management

5.1.5.1 Introduction

Using individual, on-site structural storm water controls for each development is the typical approach for controlling storm water quantity and quality. The developer finances the design and construction of these controls and, initially, is responsible for all operation and maintenance.

A potential alternative approach is for a community to install a few strategically located regional storm water controls in a subwatershed rather than require on-site controls (see Figure 5.1.5-1). For this Manual, regional storm water controls are defined as facilities designed to manage storm water runoff from multiple projects and/or properties through a local jurisdiction-sponsored program, where the individual properties may assist in the financing of the facility, and the requirement for on-site controls is either eliminated or reduced.

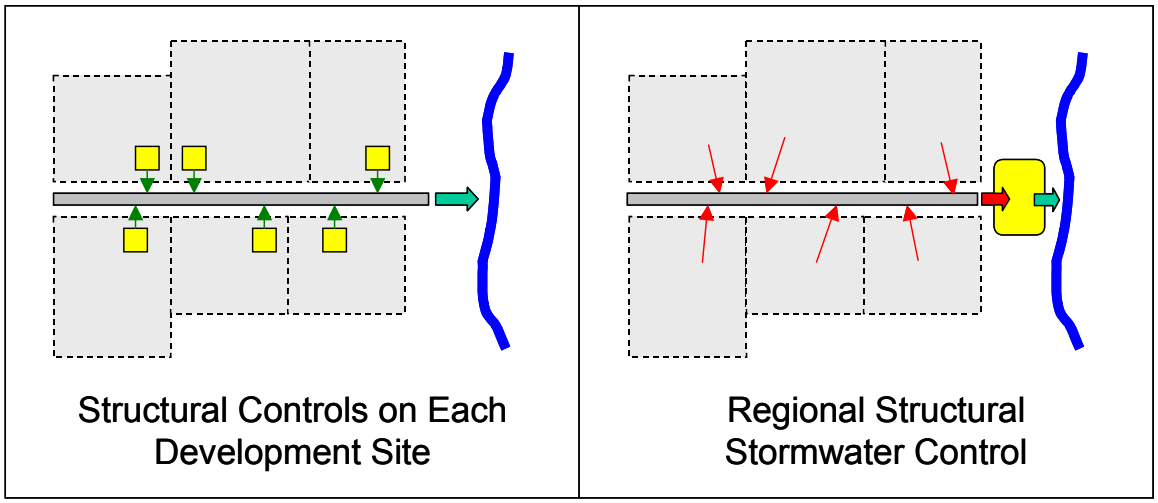


Figure 5.1.5-1 On-Site versus Regional Storm Water Management

5.1.5.2 Advantages and Disadvantages of Regional Storm Water Controls

Regional storm water facilities are significantly more cost-effective because it is easier and less expensive to build, operate, and maintain one large facility than several small ones. Regional storm water controls are generally better maintained than individual site controls because they are large, highly visible, and typically the responsibility of the local government. In addition, a larger facility poses less of a safety hazard than numerous small ones because it is more visible and is easier to secure.

There are also several disadvantages to regional storm water controls. In many cases, a community must provide capital construction funds for a regional facility, including the costs of land acquisition. However, if a downstream developer is the first to build, that person could be required to construct the facility and later be compensated by upstream developers for the capital construction costs and annual maintenance expenditures. Conversely, an upstream developer may have to establish temporary control structures if the regional facility is not in place before construction. Maintenance responsibilities generally shift from the homeowner or developer to the local government when a regional approach is selected. The local government would need to establish a storm water utility or some other program to fund and implement storm water control. Finally, a large in-stream facility can pose a greater disruption to the natural flow network and is more likely to affect wetlands within the watershed.

Below are summarized some of the “pros” and “cons” of regional storm water controls.

Advantages of Regional Storm Water Controls

- **Reduced Construction Costs** – Design and construction of a single regional storm water control facility can be far more cost-effective than numerous individual on-site structural controls.
- **Reduced Operation and Maintenance Costs** – Rather than multiple owners and associations being responsible for the maintenance of several storm water facilities on their developments, it is simpler and more cost effective to establish scheduled maintenance of a single regional facility.
- **Higher Assurance of Maintenance** – Regional storm water facilities are far more likely to be adequately maintained as they are large and have a higher visibility, and are typically the responsibility of the local government.
- **Maximum Utilization of Developable Land** – Developers would be able to maximize the utilization of the proposed development for the purpose intended by minimizing the land normally set aside for the construction of storm water structural controls.
- **Retrofit Potential** – Regional facilities can be used by a community to mitigate existing developed areas that have insufficient or no structural controls for water quality and/or quantity, as well as provide for future development.
- **Other Benefits** – Well-sited regional storm water facilities can serve as a recreational and aesthetic amenity for a community.

Disadvantages of Regional Storm Water Controls

- **Location and Siting** – Regional storm water facilities may be difficult to site, particularly for large facilities or in areas with existing development.
- **Capital Costs** – The community must typically provide capital construction funds for a regional facility, including the costs of land acquisition.
- **Maintenance** – The local government is typically responsible for the operation and maintenance of a regional storm water facility.
- **Need for Planning** – The implementation of regional storm water controls requires substantial planning, financing, and permitting. Land acquisition must be in place ahead of future projected growth.

For in-stream regional facilities:

- **Water Quality and Streambank Protection** – Without on-site water quality and streambank protection, regional controls do not protect smaller streams upstream from the facility from degradation and streambank erosion.
- **Ponding Impacts** – Upstream inundation from a regional facility impoundment can eliminate floodplains, wetlands, and other habitat.

5.1.5.3 Important Considerations for the Use of Regional Storm Water Controls

If a community decides to implement a regional storm water control, then it must ensure that the conveyances between the individual upstream developments and the regional facility can handle the design peak flows and volumes without causing adverse impact or property damage. Fully developed conditions in the regional facility drainage area should be used in the analysis.

Furthermore, unless the system consists of completely man-made conveyances (i.e. storm drains, pipes, concrete channels, etc) the on-site structural controls for water quality and downstream streambank protection will likely be required for all developments within the regional facility's drainage area. Federal water quality provisions do not allow the degradation of water bodies from untreated storm water discharges, and it is U.S. EPA policy to not allow regional storm water controls that would degrade stream quality between the upstream development and the regional facility. Further, without adequate streambank protection, aquatic habitats and water quality in the channel network upstream of a regional facility may be degraded by streambank erosion if they are not protected from bankfull flows and high velocities.

Based on these concerns, both the EPA and the U.S. Army Corps of Engineers have expressed opposition to *in-stream* regional storm water control facilities. In-stream facilities should be avoided if possible and will likely be permitted on a case-by-case basis only.

It is important to note that siting and designing regional facilities should ideally be done within a context of storm water master planning or watershed planning to be effective.

5.1.6 Using Structural Storm Water Controls in Series

5.1.6.1 Storm Water Treatment Trains

The minimum storm water management standards are an integrated planning and design approach whose components work together to limit the adverse impacts of urban development on downstream waters and riparian areas. This approach is sometimes called a storm water "treatment train." When considered comprehensively, a treatment train consists of all the design concepts and nonstructural and structural controls that work to attain water quality and quantity goals. This is illustrated in Figure 5.1.6-1.

Figure 5.1.6-1 Generalized Storm Water Treatment Train

Runoff and Load Generation – The initial part of the "train" is located at the source of runoff and pollutant load generation, and consists of *integrated* site design and pollution prevention practices that reduce runoff and storm water pollutants from the source.

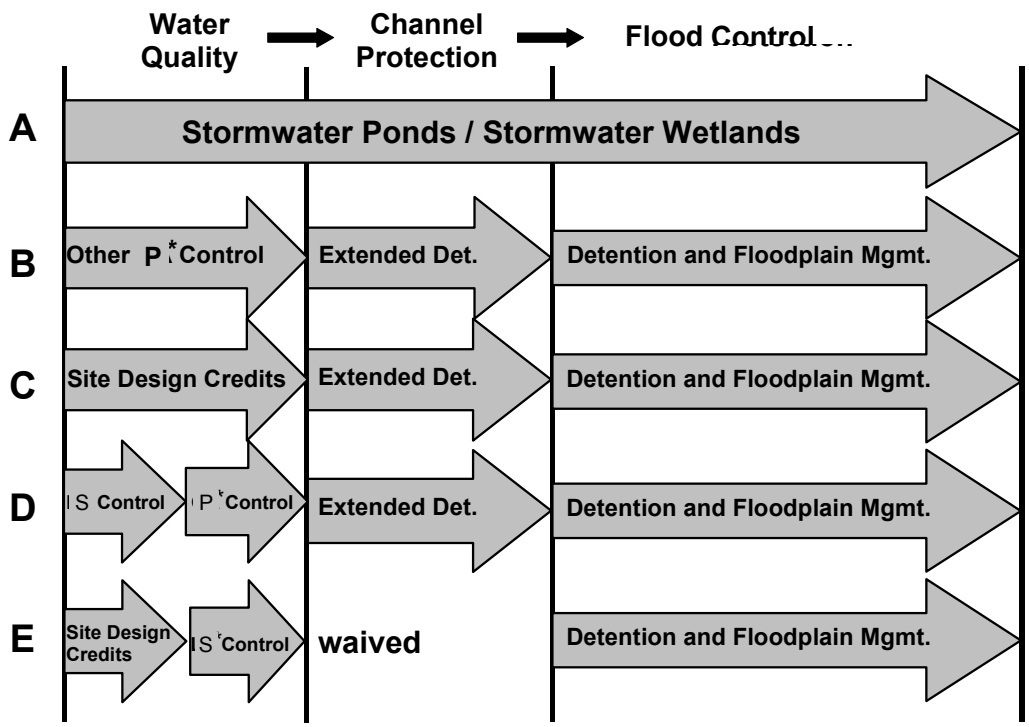
Pretreatment – The next step in the treatment train consists of pretreatment measures. These measures typically do not provide sufficient pollutant removal to meet the Primary TSS reduction goal, but do provide calculable water quality benefits that may be applied towards meeting the WQ_v treatment requirement. These measures include:

- The use of storm water *integrated* site design practices and site design credits to reduce the water quality volume (WQ_v)
- Structural controls that achieve less than the Primary TSS removal rate, but provide pretreatment
- Pretreatment facilities such as sediment forebays

Primary Treatment and/or Quantity Control – The last step is primary water quality treatment and/or quantity (streambank protection and/or flood control) control. This is achieved through the use of either a structural control to achieve both water quality and quantity benefits or a structural control to achieve water quality benefits only.

5.1.6.2 Use of Multiple Structural Controls in Series

Many combinations of structural controls in series may exist for a site. Figure 5.1.6-2 provides a number of hypothetical examples of how the *integrated* Design Approach may be addressed by using structural storm water controls.



***P - Primary Control and S - Secondary Control Limited Application.**

Figure 5.1.6-2 Examples of Structural Controls Used in Series

Referring to Figure 5.1.6-2 by line letter:

A. Two structural controls achieving Primary TSS removal each, *storm water ponds* and *storm water wetlands*, can be used to meet all of the requirements of the *integrated* Design Approach in a single facility.

B. The other structural controls achieving Primary TSS removal each (*bioretention, sand filters, infiltration trench and enhanced swale*) are typically used in combination with detention controls to meet the *integrated* Design Approach. The detention facilities are located downstream from the water quality controls either on-site or combined into a regional or neighborhood facility.

C. Line C indicates the condition where an environmentally sensitive large lot subdivision has been developed that can be designed so as to waive the water quality treatment requirement altogether. However, detention controls may still be required for downstream streambank protection and flood control.

D. Where a structural control does not meet the Primary TSS removal criteria, another downstream structural control must be added. For example, urban hotspot land may be fit or retrofit with devices adjacent to parking or service areas designed to remove petroleum hydrocarbons. These devices may also serve as pretreatment devices removing the coarser fraction of sediment. One or more downstream structural controls are then used to meet the full Primary TSS removal goal, and well as water quantity control.

E. In line E, site design credits have been employed to reduce partially the water quality volume requirement. In this case, for a smaller site, a well designed and tested structural control provides limited TSS removal while a dry detention pond handles the flooding criteria. For this location, direct discharge to a large stream and local downstream floodplain management practices have eliminated the need for streambank protection and flood control storage on site.

The combinations of structural storm water controls are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. Figures 5.1.6-3 through 5.1.6-5 illustrate the application of the treatment train concept for: a moderate density residential neighborhood, a small commercial site, and a large shopping mall site.

In Figure 5.1.6-3 rooftop runoff drains over grassed yards to backyard grass channels. Runoff from front yards and driveways reaches roadside grass channels. Finally, all storm water flows to a extended detention micropool storm water pond.

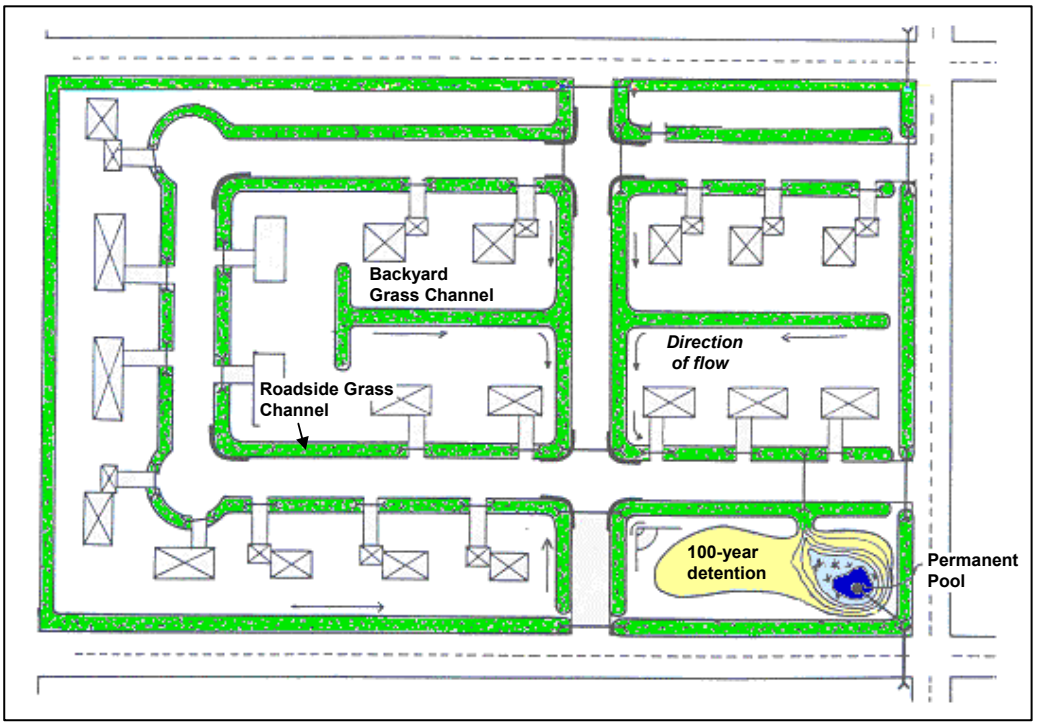


Figure 5.1.6-3 Example Treatment Train – Residential Subdivision
(Adapted from: NIPC, 2000)

A gas station and convenience store is depicted in Figure 5.1.6-4. In this case, the decision was made to intercept hydrocarbons and oils using a commercial gravity (oil-grit) separator located on the site prior to draining to perimeter sand filter for removal of finer particles and TSS. No storm water control for streambank protection is required as the system drains to the municipal storm drain pipe system. Flood control is provided by a regional storm water control downstream.

Figure 5.1.6-5 shows an example treatment train for a commercial shopping center. In this case, runoff from rooftops and parking lots drains to depressed parking lot islands, perimeter grass channels, and bioretention areas. Slotted curbs are used at the entrances to these swales to better distribute the flow and to settle out the very coarse particles at the parking lot edge for sweepers to remove. Runoff is then conveyed to an extended detention wet pond for additional pollutant removal and streambank protection. Flood control is provided through parking lot detention.

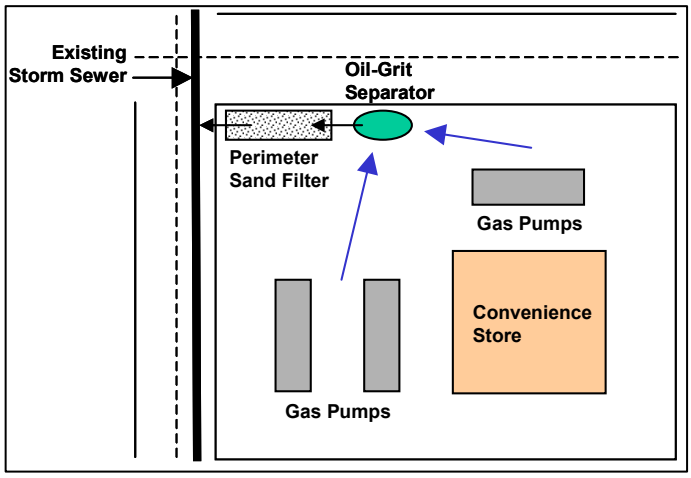


Figure 5.1.6-4 Example Treatment Train – Small Commercial Site

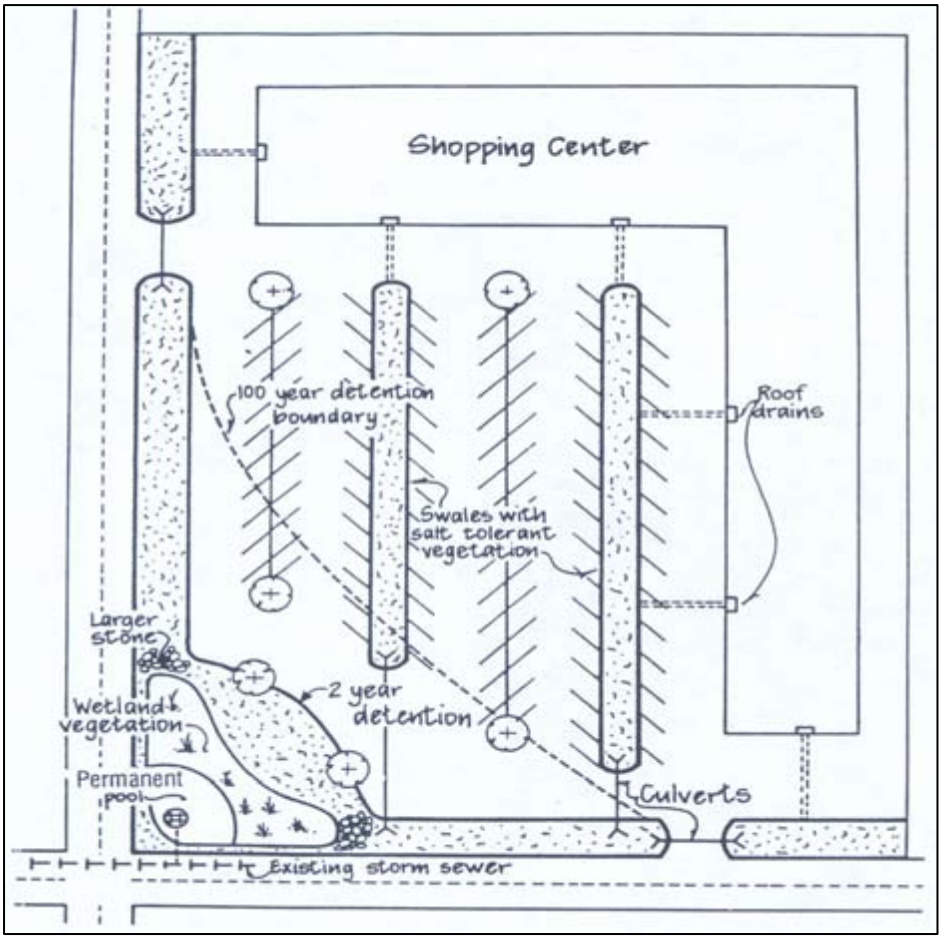


Figure 5.1.6-5 Example Treatment Train – Large Commercial Development
(Source: NIPC, 2000)

5.1.6.3 Calculation of Pollutant Removal for Structural Controls in Series

For two or more structural storm water controls used in combination, it is often important to have an estimate of the pollutant removal efficiency of the treatment train. Pollutant removal rates for structural controls in series are not additive. For pollutants in particulate form, the actual removal rate (expressed in terms of percentage of pollution removed) varies directly with the pollution concentration and sediment size distribution of runoff entering a facility.

For example, a storm water pond facility will have a much higher pollutant removal percentage for very turbid runoff than for clearer water. When two storm water ponds are placed in series, the second pond will treat an incoming particulate pollutant load very different from the first pond. The upstream pond captures the easily removed larger sediment sizes, passing on an outflow with a lower concentration of TSS but with a higher proportion of finer particle sizes. Hence, the removal capability of the second pond for TSS is considerably less than the first pond. Recent findings suggest that the second pond in series can provide as little as half the removal efficiency of the upstream pond.

To estimate the pollutant removal rate of structural controls in series, a method is used in which the removal efficiency of a downstream structural control is reduced to account for the pollutant removal of the upstream control(s). The following steps are used to determine the pollutant removal:

- For each drainage area, list the structural controls in order, upstream to downstream, along with their expected average pollutant removal rates from Table 5.1.2-1 for the pollutants of concern.
- For any structural control with a Primary TSS removal rate located downstream from another control that has an equivalent TSS removal rate, the designer should use 50% of the normal pollutant removal rate for the second control in series. For any structural control with a Primary TSS removal rate located downstream from a control that cannot achieve the Primary TSS reduction goal, the designer should use 75% of the normal pollutant removal rate for the second control in series.

  For example, if a structural control has a Primary TSS removal rate, then a 35% to 40% TSS removal rate would be assumed for this control if it were placed downstream from another equivalent control in the treatment train (0.5 x 70% to 80%). If it were placed downstream from a structural control that cannot achieve the Primary TSS reduction goal, a 52.5% to 60% TSS removal rate would be assumed (0.75 x 70% to 80%). Use this rule with caution depending on the actual pollutant of concern and make allowance for differences among structural control pollutant removal rates for different pollutants. Actual data from similar situations should be used to temper or override this rule of thumb where available.
- For cases where a structural control which cannot achieve a Primary TSS removal rate is sited upstream from a structural control which can achieve the 70% to 80% removal in the treatment train, the downstream structural control is given full credit for removal of pollutants.
- Apply the following equation for calculation of approximate total accumulated pollution removal for controls in series:

  Final Pollutant Removal = (Total load * Control1 removal rate) + (Remaining load * Control2 removal rate) + … for other Controls in series.

Example

TSS is the pollutant of concern and a commercial device is inserted that has a 20% sediment removal rate. A storm water pond is designed at the site outlet. A second storm water pond is located downstream from the first one in series. What is the total TSS removal rate? The following information is given:

Control 1 (Commercial Device) = 20% TSS removal

Control 2 (Storm water Pond 1) = 80% TSS removal (use 1.0 x design removal rate)

Control 3 (Storm water Pond 2) = 40% TSS removal (use 0.5 x design removal rate)

Then applying the controls in order and working in terms of "units" of TSS starting at 100 units:

For Control 1: 100 units of TSS * 20% removal rate = 20 units removed

100 units - 20 units removed = 80 units of TSS remaining

For Control 2: 80 units of TSS * 80% removal rate = 64 units removed

80 units - 64 units removed = 16 units of TSS remaining

For Control 3: 16 units of TSS * 40% removal rate = 6 units removed

16 units - 6 units removed = 10 units TSS remaining

For the treatment train in total = 100 units TSS – 10 units TSS remaining = **90% removal**

5.1.6.4 Routing with WQ_v Removed

When off-line structural controls such as bioretention areas, sand filters and infiltration trenches capture and remove the water quality volume (WQ_v), downstream structural controls do not have to account for this volume during design. That is, the WQ_v may be subtracted from the total volume that would otherwise need to be routed through the downstream structural controls.

From a calculation standpoint this would amount to removing the initial WQ_v from the beginning of the runoff hydrograph – thus creating a "notch" in the runoff hydrograph. Since most commercially available hydrologic modeling packages cannot handle this type of action, the following method to adjust "CN" values has been created to facilitate removal from the runoff hydrograph of approximately the WQ_v:

- Enter the horizontal axis on Figure 5.1.6-6 with the impervious percentage of the watershed and read upward to the predominant soil type (interpolation between curves is permitted)
- Read left to the factor
- Multiply the curve number for the sub-watershed that includes the water quality basin by this factor – this provides a smaller curve number

The difference in curve number will generate a runoff hydrograph that has a volume less than the original volume by an amount approximately equal to the WQ_v. This method should be used only for bioretention areas, filter facilities, and infiltration trenches where the drawdown time is ≥ 24 hours.

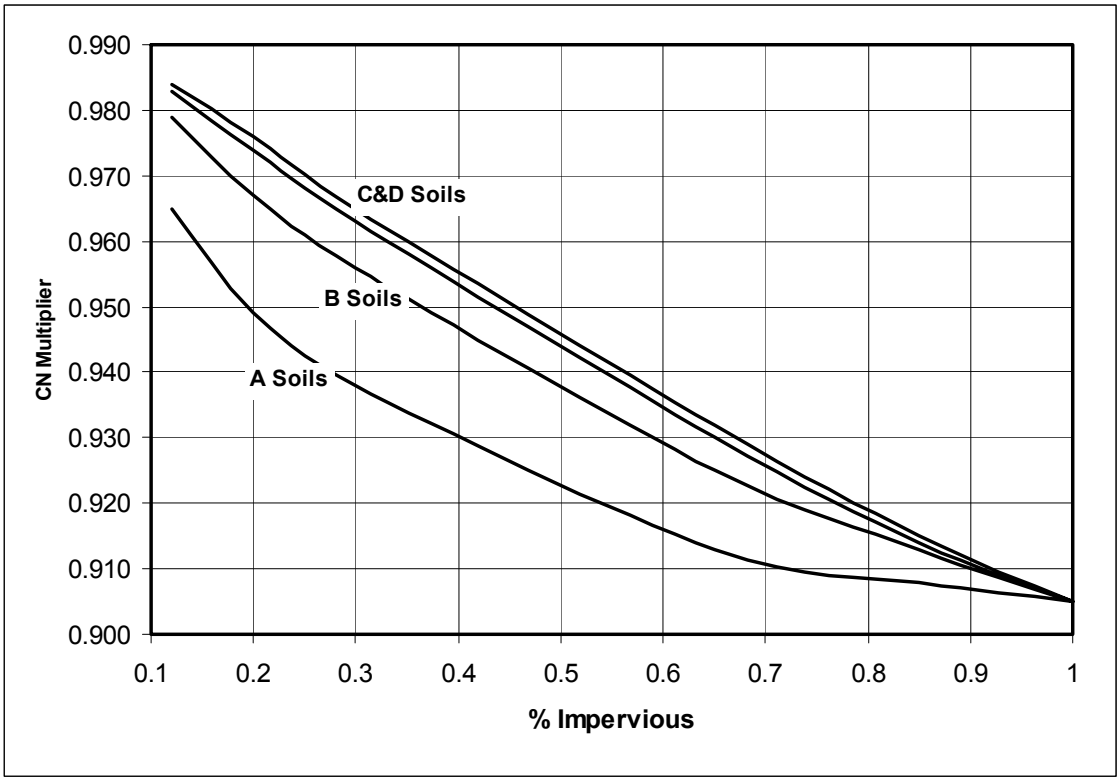


Figure 5.1.6-6 Curve Number Adjustment Factor

Example

A site design employs an infiltration trench for the WQ_v and has a curve number of 72, is B type soil, and has an impervious percentage of 60%, the factor from Figure 5.1.6-6 is 0.93. The curve number to be used in calculation of a runoff hydrograph for the quantity controls would be: (72*0.93) = 67.

Section 5.2
Storm Water Controls

5.2.1 Bioretention Areas

Structural Storm Water Control

Description: Shallow storm water basin or landscaped area that utilizes engineered soils and vegetation to capture and treat runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum contributing drainage area of 5 acres (< 2 acres recommended)
- Often located in "landscaping islands"
- Treatment area consists of grass filter, sand bed, ponding area, organic/mulch layer, planting soil, and vegetation
- Typically requires 5 feet of head

ADVANTAGES / BENEFITS:

- Applicable to small drainage areas
- Good for highly impervious areas, flexible siting
- Good retrofit capability
- Relatively low maintenance requirements
- Can be planned as an aesthetic feature

DISADVANTAGES / LIMITATIONS:

- Requires extensive landscaping if in public area
- Not recommended for areas with steep slopes

MAINTENANCE REQUIREMENTS:

- Inspect and repair/replace treatment area components

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/50%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
M	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No Data	**Pathogens** - Coliform, Streptococci, E. Coli removal

STORM WATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
S	**Streambank Protection**
S	**On-Site Flood Control**
	Downstream Flood Control

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

S - in certain situations

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
M	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max. (< 2 acres recommended)*

Soils: *Planting soils must meet specified criteria; No restrictions on surrounding soils*

Other Considerations:

- *Use of native plants is recommended*

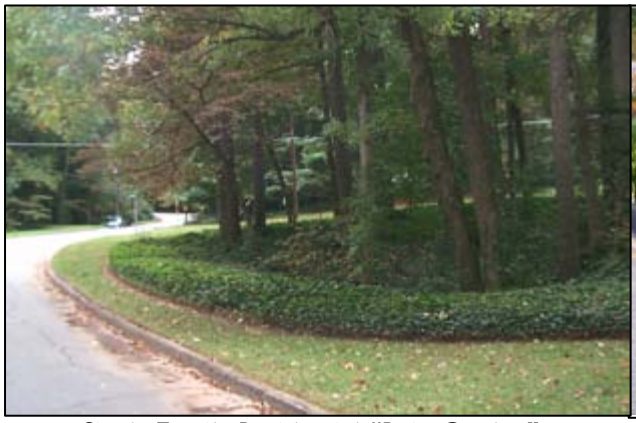
L = Low M = Moderate H = High

5.2.1.1 General Description

Bioretention areas (also referred to as *bioretention filters* or *rain gardens*) are structural storm water controls that capture and temporarily store the water quality protection volume (WQ_v) using soils and vegetation in shallow basins or landscaped areas to remove pollutants from storm water runoff.

Bioretention areas are engineered facilities in which runoff is conveyed as sheet flow to the "treatment area" which consists of a grass buffer strip, ponding area, organic or mulch layer, planting soil, and vegetation. An optional sand bed can also be included in the design to provide aeration and drainage of the planting soil. The filtered runoff is typically collected and returned to the conveyance system, though it can also infiltrate into the surrounding soil in areas with porous soils.

There are numerous design applications, both on- and off-line, for bioretention areas. These include use on single-family residential lots (*rain gardens*), as off-line facilities adjacent to parking lots, along highway and road drainage swales, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments. Figures 5.2.1-1 and 5.2.1-2 illustrate a number of examples of bioretention facilities in both photographs and drawings.



Single-Family Residential "Rain Garden"

Landscaped Island

Newly Constructed Bioretention Area

Newly Planted Bioretention Area After Storm

Figure 5.2.1-1 Bioretention Area Examples

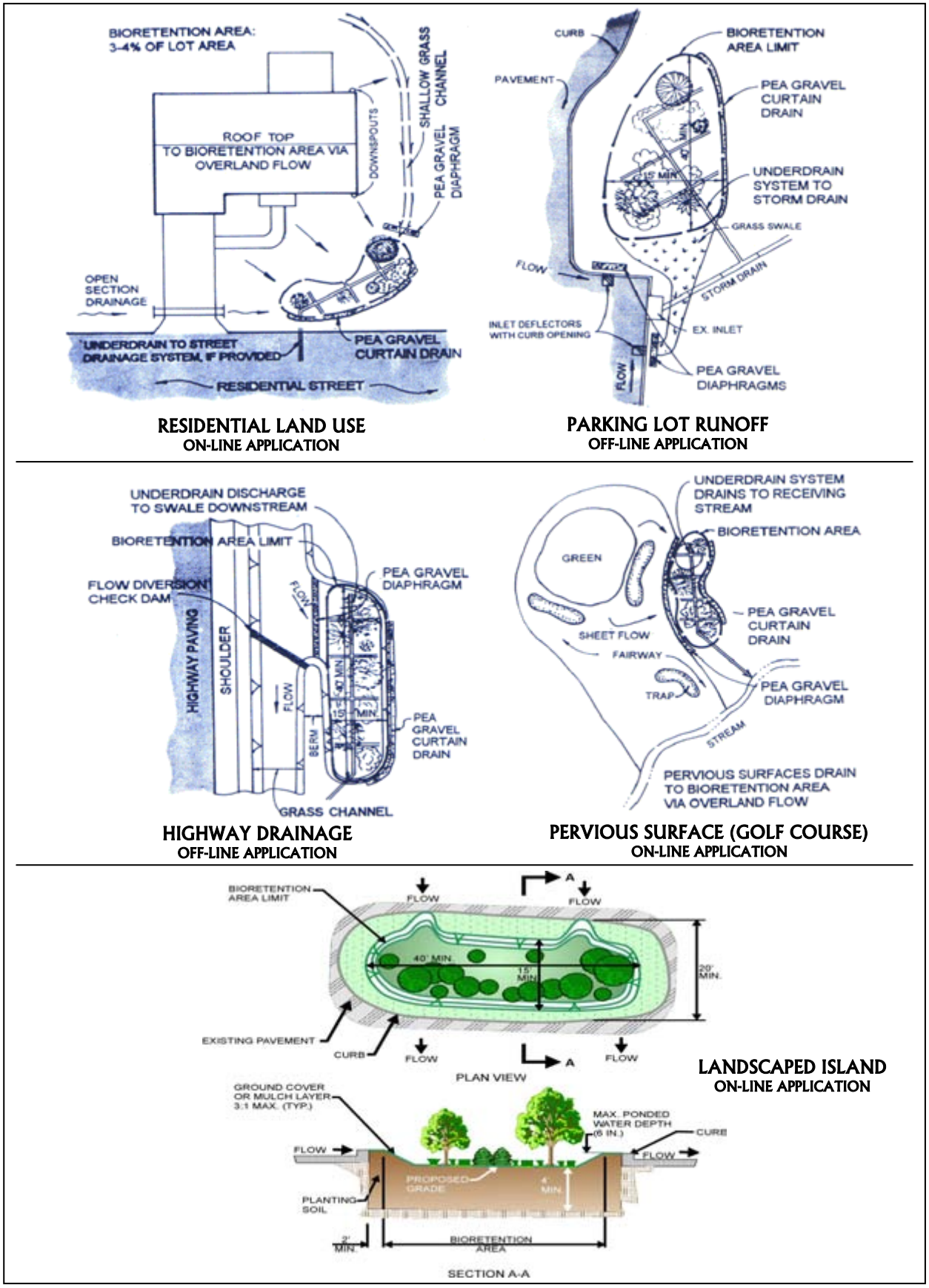


Figure 5.2.1-2 Bioretention Area Applications
(Source: Claytor and Schueler, 1996)

5.2.1.2 Storm Water Management Suitability

Bioretention areas are designed primarily for storm water quality, i.e. the removal of storm water pollutants. Bioretention can provide limited runoff quantity control, particularly for smaller storm events. These facilities may sometimes be used to partially or completely meet streambank protection requirements on smaller sites. However, bioretention areas will typically need to be used in conjunction with another structural control to provide streambank protection as well as flood control. It is important to ensure that a bioretention area safely bypasses higher flows.

Water Quality Protection

Bioretention is an excellent storm water treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the bioretention area is designed to perform a specific function (see Figure 5.2.1-3 on page 5.2-12). The *grass filter strip (or grass channel)* reduces incoming runoff velocity and filters particulates from the runoff. The *ponding area* provides for temporary storage of storm water runoff prior to its evaporation, infiltration, or uptake and provides additional settling capacity. The *organic or mulch layer* provides filtration as well as an environment conducive to the growth of microorganisms that degrade hydrocarbons and organic material. The *planting soil* in the bioretention facility acts as a filtration system, and clay in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients, and other pollutants. Both *woody and herbaceous plants* in the ponding area provide vegetative uptake of runoff and pollutants and also serve to stabilize the surrounding soils. Finally, an optional *sand bed* provides for positive drainage and aerobic conditions in the planting soil and provides a final polishing treatment media.

Section 5.2.1.3 gives data on pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

For smaller sites, a bioretention area may be designed to capture the entire streambank protection volume SP_v in either an off- or on-line configuration. Given that a bioretention facility is typically designed to completely drain over 48 hours, the requirement of extended detention of the 1-year, 24-hour storm runoff volume will be met. For larger sites where only the WQ_v is diverted to the bioretention facility, another structural control must be used to provide SP_v extended detention.

Flood Control

Bioretention areas must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the ponding area, mulch layer, and vegetation.

Credit for the volume of runoff removed and treated in the bioretention area may be taken in flood control calculations (see Section 5.1).

5.2.1.3 Pollutant Removal Capabilities

Bioretention areas are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed bioretention areas can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 60%**

- **Total Nitrogen – 50%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 80%**

For additional information and data on pollutant removal capabilities for bioretention areas, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Storm Water Best Management Practices (BMP) Database at www.bmpdatabase.org

5.2.1.4 Application and Site Feasibility Criteria

Bioretention areas are suitable for many types of development, from single-family residential to high-density commercial projects. Bioretention is also well suited for small lots, including those of one acre or less. Because of its ability to be incorporated in landscaped areas, the use of bioretention is extremely flexible. Bioretention areas are an ideal structural storm water control for use as roadway median strips and parking lot islands and are also good candidates for the treatment of runoff from pervious areas, such as a golf course. Bioretention can also be used to retrofit existing development with storm water quality treatment capacity.

The following criteria should be evaluated to ensure the suitability of a bioretention area for meeting storm water management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Storm Water Control – NO
- Hot Spot - YES

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum; 0.5 to 2 acres are preferred.
- Space Required – Approximately 5-7% of the tributary impervious area is normally required.
- Site Slope – No more than 6% slope
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 3-5 feet
- Minimum Depth to Water Table – A separation distance of 2 feet recommended between the bottom of the bioretention facility and the elevation of the seasonally high water table.
- Soils – No restrictions; engineered media required

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

5.2.1.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a bioretention facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- Bioretention areas should have a maximum contributing drainage area of 5 acres or less; 0.5 to 2 acres are preferred. Multiple bioretention areas can be used for larger areas.

- Bioretention areas can either be used to capture sheet flow from a drainage area or function as an off-line device. On-line designs should be limited to a maximum drainage area of 0.5 acres unless special precautions are taken to protect from erosion during high flows.
- When used in an off-line configuration, the water quality protection volume (WQ_v) is diverted to the bioretention area through the use of a flow splitter. Storm water flows greater than the WQ_v are diverted to other controls or downstream (see Section 5.1 for more discussion of off-line systems and design guidance for diversion structures and flow splitters).
- Bioretention systems are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.
- Bioretention area locations should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure that the desired runoff flow enters the facility with no more than the maximum design depth.

B. GENERAL DESIGN

- A well-designed bioretention area consists of:

  1 **Grass filter strip (or grass channel) between the contributing drainage area and the ponding area, except where site conditions preclude its use,**

  2 **Ponding area containing vegetation with a planting soil bed,**

  3 **Organic/mulch layer,**

  4 **Pea gravel layer between the planting soil and the gravel underneath to provide filtering of the particles prior to entering gravel layer,**

  5 **Gravel and perforated pipe underdrain system to collect runoff that has filtered through the soil layers (bioretention areas can optionally be designed to infiltrate into the soil – see description of infiltration trenches for infiltration criteria).**

- A bioretention area design will also include some of the following:

  1 Optional **sand filter layer** to spread flow, filter runoff, and aid in aeration and drainage of the planting soil.

  2 **Stone diaphragm** at the beginning of the grass filter strip to reduce runoff velocities and spread flow into the grass filter.

  3 **Inflow diversion** or an **overflow structure** consisting of one of five main methods:

  - Use a flow diversion structure
  - For curbed pavements use an inlet deflector (see Figure 5.2.1-6).
  - Use a slotted curb and design the parking lot grades to divert the WQ_v into the facility. Bypass additional runoff to a downstream catch basin inlet. Requires temporary ponding in the parking lot (see Figure 5.2.1-5).
  - Figure 5.2.1-2c illustrates the use of a short deflector weir (maximum height 6 inches) designed to divert the maximum water quality peak flow into the bioretention area.
  - An in-system overflow consisting of an overflow catch basin inlet and/or a pea gravel curtain drain overflow.

See Figure 5.2.1-3 for an overview of the various components of a bioretention area. Figure 5.2.1-4 provides a plan view and profile schematic of an on-line bioretention area. An example of an off-line facility is shown in Figure 5.2.1-5.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- Recommended minimum dimensions of a bioretention area are 10 feet wide by 40 feet long. All designs except small residential applications should maintain a length to width ratio of at least 2:1.
- The planting soil filter bed is sized using a Darcy's Law equation with a filter bed drain time of less than 48 hours (less than 6 hours residential neighborhoods and 24 hours non-residential preferred) and a coefficient of permeability (k) of greater than 0.5 ft/day.
- The maximum recommended ponding depth of the bioretention areas is 6 inches with a drain time normally of 3 to 4 hours in residential settings.
- The planting soil bed must be at least 2.5 feet in depth and up to 4 feet if large trees are to be planted. Planting soils should be sandy loam, loamy sand, or loam texture with a clay content ranging from 5 to 8%. The soil must have an infiltration rate of at least 0.5 inches per hour (1.0 in/hr preferred) and a pH between 5.5 and 6.5. In addition, the planting soil should have a 1.5 to 3% organic content and a maximum 500 ppm concentration of soluble salts.
- For on-line configurations, a grass filter strip with a pea gravel diaphragm is typically utilized (see Figure 5.2.1-3) as the pretreatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Design guidance on filter strips for pretreatment can be found in subsection 5.2.12 (*Filter Strip)*.
- For off-line applications, a grass channel with a pea gravel diaphragm flow spreader is used for pretreatment. The length of the grass channel depends on the drainage area, land use, and channel slope. The minimum grassed channel length should be 20 feet. Design guidance on grass channels for pretreatment can be found in subsection 5.2.3 (*Grass Channel)*.
- The mulch layer should consist of 2 to 4 inches of commercially available fine shredded hardwood mulch or shredded hardwood chips.
- The sand bed (optional) should be 12 to 18 inches thick. Sand should be clean and have less than 15% silt or clay content.
- Pea gravel for the 4" to 9" thick layer above the gravel bedding (and diaphragm and curtain, where used), should be ASTM D 448 size No. 6 ($^1/_8$" to ¼").
- The underdrain collection system is equipped with a 6-inch perforated PVC pipe (AASHTO M 252) in an 8-inch gravel layer. The pipe should have 3/8-inch perforations, spaced at 6-inch centers, with a minimum of 4 holes per row. The pipe is spaced at a maximum of 10 feet on center and a minimum grade of 0.5% must be maintained.
- A narrow 24" wide permeable filter fabric is placed between the gravel layer and the pea gravel layer directly above the perforated pipes to limit piping of soil directly into the pipe. Filter fabric is also placed along the vertical or sloping outer walls of the bioretention system to limit vertical infiltration prior to filtration through the soil.

D. PRETREATMENT / INLETS

- Adequate pretreatment and inlet protection for bioretention systems is provided when all of the following are provided: (a) grass filter strip below a level spreader, or grass channel, (b) pea gravel diaphragm and (c) an organic or mulch layer.

E. OUTLET STRUCTURES

- Outlet pipe is to be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary.

F. EMERGENCY SPILLWAY

- An overflow structure and nonerosive overflow channel must be provided to safely pass flows from the bioretention area that exceeds the storage capacity to a stabilized downstream area or watercourse. If the system is located off-line, the overflow should be set above the shallow ponding limit.
- The high flow overflow system within the structure consists of a yard drain catchbasin (Figure 5.2.1-3), though any number of conventional systems could be used. The throat of the catch basin inlet is normally placed 6 inches above the mulch layer. It should be designed as a domed grate or a covered weir structure to avoid clogging with floatation mulch and debris, and should be located at a distance from inlets to avoid short circuiting of flow. It may also be placed into the side slope of the structure maintaining a neat contoured appearance.

G. MAINTENANCE ACCESS

- Adequate access must be provided for all bioretention facilities for inspection, maintenance, and landscaping upkeep. Appropriate equipment and vehicles are essential.

H. SAFETY FEATURES

- Bioretention areas generally do not require any special safety features. Fencing of bioretention facilities is not generally desirable.

I. LANDSCAPING

- Landscaping is critical to the performance and function of bioretention areas.
- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the facility. Side slopes should be sodded to limit erosion of fine particles onto the bioretention surface.
- The bioretention area should be vegetated to resemble a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, scrub layer, and herbaceous ground cover. Three species each of both trees and scrubs are recommended to be planted.
- The tree-to-shrub ratio should be 2:1 to 3:1. On average, the trees should be spaced 8 feet apart. Plants should be placed at regular intervals to replicate a natural forest. Woody vegetation should not be specified at inflow locations.
- After the trees and shrubs are established, the ground cover and mulch should be established.
- Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost aesthetics, maintenance, etc. Planting recommendations for bioretention facilities are as follows:
  - Native plant species should be specified over non-native species.
  - Vegetation should be selected based on a specified zone of hydric tolerance.
  - A selection of trees with an understory of shrubs and herbaceous materials should be provided.

Additional information and guidance on the appropriate woody and herbaceous species appropriate for bioretention in North Central Texas, and their planting and establishment, can be found in Appendix F, *Landscaping and Aesthetics Guidance*.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of bioretention areas may be limited by low head
- High Relief – Ponding area surface must be relatively level

- Karst – Use poly-liner or impermeable membrane to seal bottom

Soils

- No restrictions

Special Downstream Watershed Considerations

- Aquifer Protection – No restrictions, if designed with no infiltration (i.e. outflow to groundwater)

5.2.1.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Approach

Calculate the Water Quality Protection Volume (WQ_v), Streambank Protection Volume (SP_v), and the 100-Year Flood Discharge (Q_f).

Details on the *integrated* Design Approach are found in Section 1.2.

Step 2 Determine if the development site and conditions are appropriate for the use of a bioretention area

Consider the Application and Site Feasibility Criteria in subsections 5.2.1.4 and 5.2.1.5-A (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 5.2.1.5-J (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see subsection 2.1.9).

(a) Using WQ_v (or total volume to be captured), compute CN

(b) Compute time of concentration using TR-55 method

(c) Determine appropriate unit peak discharge from time of concentration

(d) Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5 Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the bioretention area.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6 Determine size of bioretention ponding/filter area

The required planting soil filter bed area is computed using the following equation (based on Darcy's Law):

$$\mathbf{A_f = (WQ_v)\,(d_f) \,/\, [\,(k)\,(h_f + d_f)\,(t_f)]}$$

where:

A_f = surface area of ponding area (ft^2)

WQ_v = water quality protection volume (or total volume to be captured)

d_f = filter bed depth (2.5 feet minimum)

k = coefficient of permeability of filter media (ft/day) (use 0.5 ft/day for silt-loam)

h_f = average height of water above filter bed (ft) (typically 3 inches, which is half of

the 6-inch ponding depth)

t_f = design filter bed drain time (days) (2.0 days or 48 hours is recommended maximum)

Step 7 Set design elevations and dimensions of facility

See subsection 5.2.1.5-C (Physical Specifications/Geometry).

Step 8 Design conveyances to facility (off-line systems)

See the example figures to determine the type of conveyances needed for the site.

Step 9 Design pretreatment

Pretreat with a grass filter strip (on-line configuration) or grass channel (off-line), and stone diaphragm.

Step 10 Size underdrain system

See subsection 5.2.1.5-C (Physical Specifications/Geometry)

Step 11 Design emergency overflow

An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. Nonerosive velocities need to be ensured at the outlet point.

Step 12 Prepare Vegetation and Landscaping Plan

A landscaping plan for the bioretention area should be prepared to indicate how it will be established with vegetation.

See subsection 5.2.1.5-I (Landscaping) and Appendix F for more details.

See Appendix D-2 for a Bioretention Area Design Example

5.2.1.7 Inspection and Maintenance Requirements

Table 5.2.1-1 Typical Maintenance Activities for Bioretention Areas

Activity	Schedule
• Pruning and weeding to maintain appearance. • Mulch replacement when erosion is evident. • Remove trash and debris.	As needed
• Inspect inflow points for clogging (off-line systems). Remove any sediment. • Inspect filter strip/grass channel for erosion or gullying. Re-seed or sod as necessary. • Trees and shrubs should be inspected to evaluate their health and remove any dead or severely diseased vegetation.	Semi-annually
• The planting soils should be tested for pH to establish acidic levels. If the pH is below 5.2, limestone should be applied. If the pH is above 7.0 to 8.0, then iron sulfate plus sulfur can be added to reduce the pH.	Annually
• Replace mulch over the entire area. • Replace pea gravel diaphragm if warranted (or when the voids are obviously filled with sediment and water is no longer infiltrating).	2 to 3 years

(Source: EPA, 1999)

Additional Maintenance Considerations and Requirements

- The surface of the ponding area may become clogged with fine sediment over time. Core aeration or cultivating of unvegetated areas may be required to ensure adequate filtration.

Regular inspection and maintenance is critical to the effective operation of bioretention facilities as designed. Maintenance responsibility for a bioretention area should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

5.2.1.8 Example Schematics

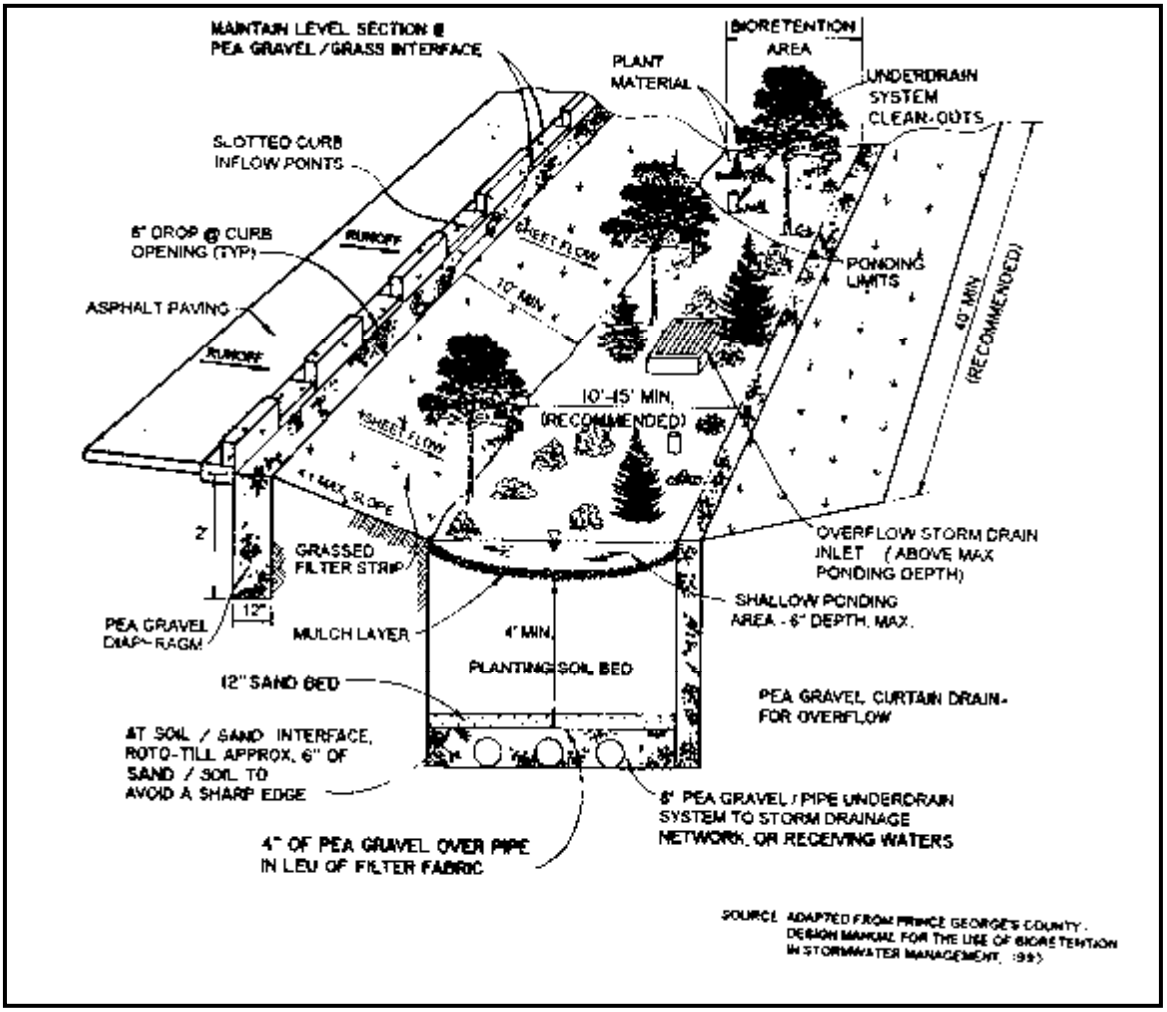


Figure 5.2.1-3 Schematic of a Typical Bioretention Area
(Source: Claytor and Schueler, 1996)

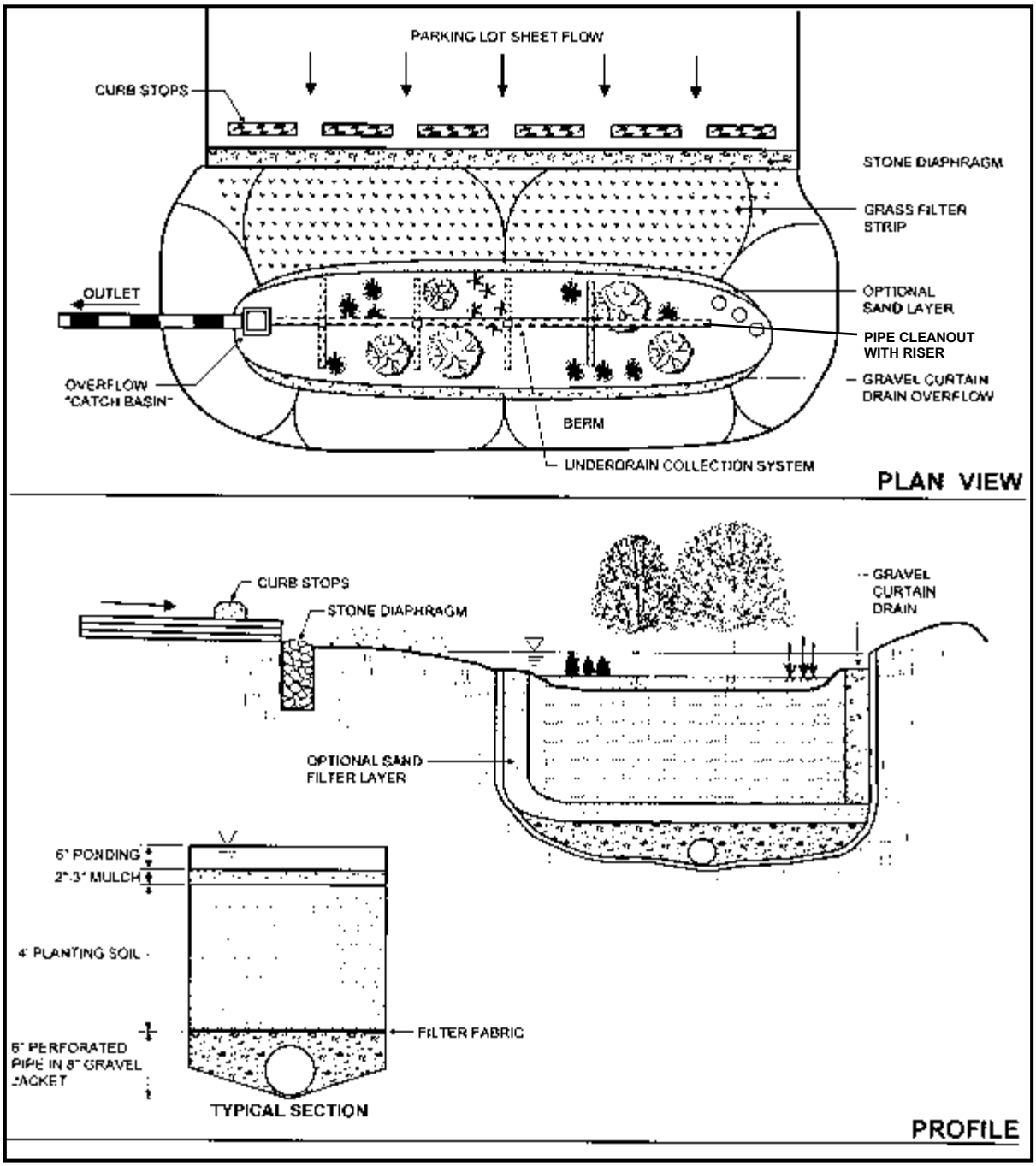


Figure 5.2.1-4 Schematic of a Typical On-Line Bioretention Area
(Source: Claytor and Schueler, 1996)

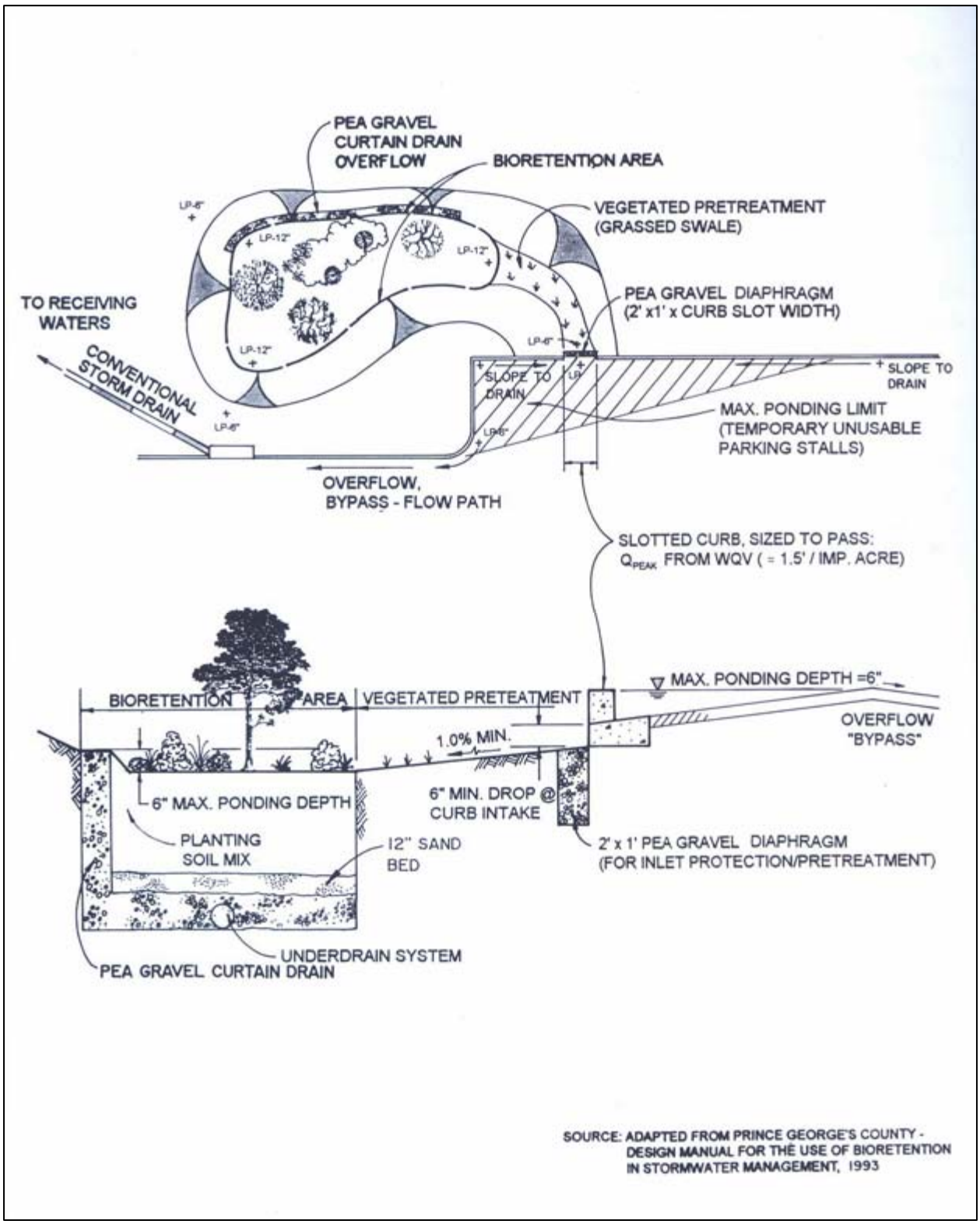


Figure 5.2.1-5 Schematic of a Typical Off-Line Bioretention Area
(Source: Claytor and Schueler, 1996)

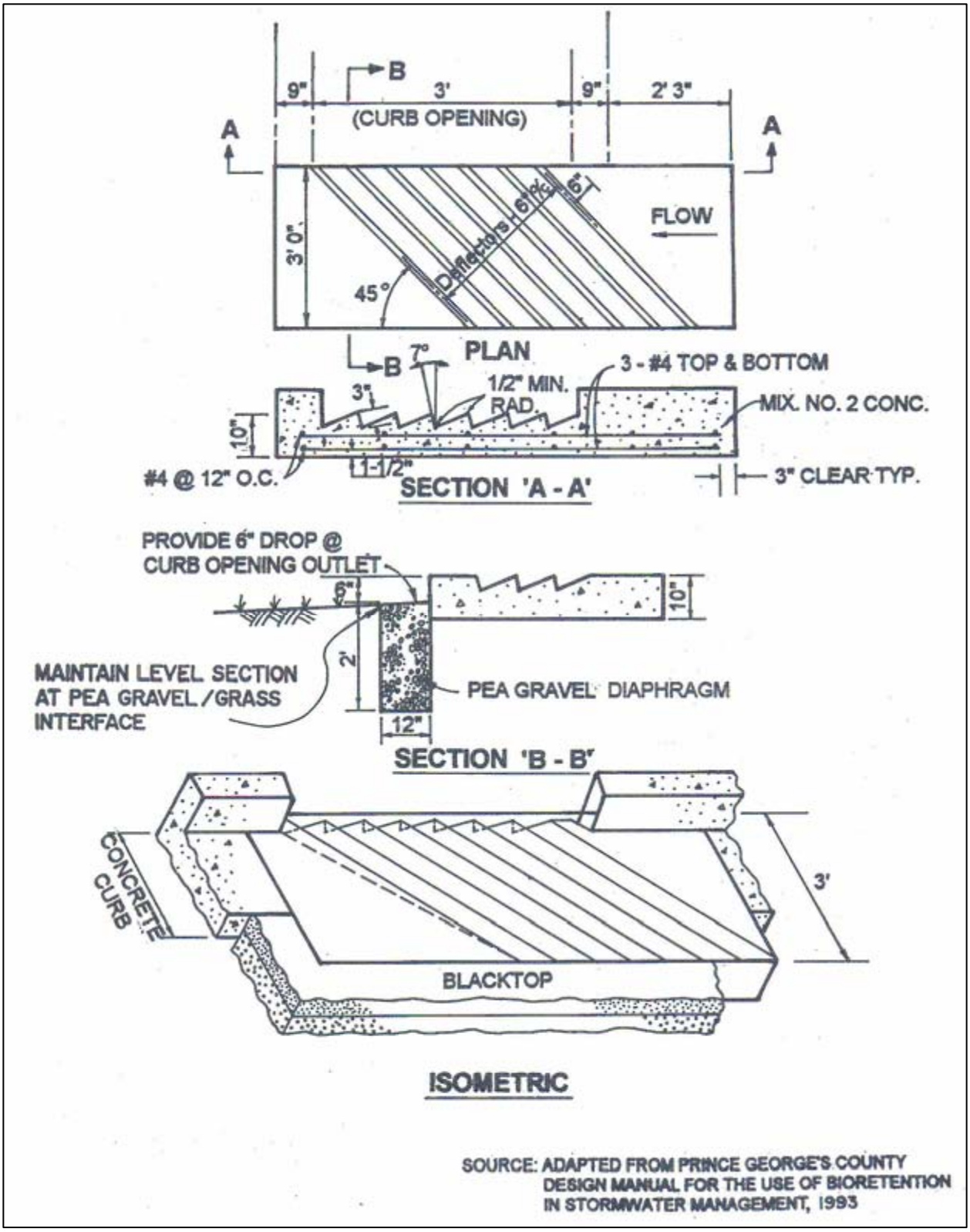


Figure 5.2.1-6 Schematic of a Typical Inlet Deflector
(Source: Claytor and Schueler, 1996)

Design Procedure Form: Bioretention Areas

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements Compute Runoff Coefficient, R_v Compute WQ_v	R_v = _____ WQ_v = _____ acre-ft
1b. Compute SP_v Compute average release rate Compute (as necessary) Q_f	SP_v = _____ acre-ft release rate = _____ cfs Q_f = _____ cfs
BIORETENTION AREA DESIGN	
2. Is the use of a bioretention area appropriate?	***See subsections 5.2.1.4 and 5.2.1.5 - A***
3. Confirm local design criteria and applicability	
4. Determine size of bioretention filter area	A_f = _____ ft^2
5. Set design elevations and dimensions	Length = _____ ft Width = _____ ft _____ elevation top of facility _____ other elev: _____ _____ other elev: _____ _____ other elev: _____
6. Conveyance to bioretention facililty	_____Online or _____ Offline?
7. Pretreatment	Type: _____
8. Size underdrain area Based on guidance: Approx. 10% A_f	Length = _____ ft
9. Overdrain design	Type: _____ Size: _____
10. Emergency storm weir design Overflow weir - Weir equation	Length = _____ ft
11. Choose plants for planting area	*Select native plants based on resistance to drought and inundation, cost, aesthetics, maintenance, etc.* ***See Appendix F***

Notes: _____

5.2.2 Enhanced Swales

General Application
Structural Storm Water Control

Description: Vegetated open channels that are explicitly designed and constructed to capture and treat storm water runoff within dry or wet cells formed by check dams or other means.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Longitudinal slopes must be less than 4%
- Bottom width of 2 to 8 feet
- Side slopes 2:1 or flatter; 4:1 recommended
- Convey the "Conveyance" storm event with minimum freeboard, as specified in Local Criteria

ADVANTAGES / BENEFITS:

- Combines storm water treatment with runoff conveyance system
- Less expensive than curb and gutter
- Reduces runoff velocity

DISADVANTAGES / LIMITATIONS:

- Higher maintenance than curb and gutter systems
- Cannot be used on steep slopes
- Possible resuspension of sediment
- Potential for odor / mosquitoes (wet swale)
- Concerns with aesthetics of 4"-6" high grass in residential areas

MAINTENANCE REQUIREMENTS:

- Maintain grass heights of approximately 4 to 6 inches (dry swale)
- Remove sediment from forebay and channel

POLLUTANT REMOVAL (DRY SWALE)

80%	**Total Suspended Solids**
25/40%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
40%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- P **Water Quality Protection**
- S **Streambank Protection**
- S **On-Site Flood Control**
- S **Downstream Flood Control**

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

IMPLEMENTATION CONSIDERATIONS

- H **Land Requirement**
- M **Capital Cost**
- L **Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Permeable soil layer (dry swale)*
- *Wetland plants (wet swale)*

L=Low **M**=Moderate **H**=High

5.2.2.1 General Description

Enhanced swales (also referred to as *vegetated open channels* or *water quality swales*) are conveyance channels engineered to capture and treat the water quality volume (WQ_v) for a drainage area. They differ from a normal drainage channel or swale through the incorporation of specific features that enhance storm water pollutant removal effectiveness.

Enhanced swales are designed with limited longitudinal slopes to force the flow to be slow and shallow, thus allowing for particulates to settle and limiting the effects of erosion. Berms and/or check dams installed perpendicular to the flow path promote settling and infiltration.

There are two primary enhanced swale designs, the *dry swale* and the *wet swale* (or *wetland channel*). Below are descriptions of these two designs:

- **Dry Swale** – The dry swale is a vegetated conveyance channel designed to include a filter bed of prepared soil that overlays an underdrain system. Dry swales are sized to allow the entire WQ_v to be filtered or infiltrated through the bottom of the swale. Because they are dry most of the time, they are often the preferred option in residential settings.
- **Wet Swale (Wetland Channel)** – The wet swale is a vegetated channel designed to retain water or marshy conditions that support wetland vegetation. A high water table or poorly drained soils are necessary to retain water. The wet swale essentially acts as a linear shallow wetland treatment system, where the WQ_v is retained.

Enhanced Dry Swale

Enhanced Dry Swale

Figure 5.2.2-1 Enhanced Swale Examples

Dry and wet swales are not to be confused with a *filter strip* or *grass channel*, which are Limited Application structural controls and not considered acceptable for meeting the TSS removal performance goal by themselves. Ordinary *grass channels* are not engineered to provide the same treatment capability as a well-designed dry swale with filter media. *Filter strips* are designed to accommodate overland flow rather than channelized flow and can be used as storm water credits to help reduce the total water quality treatment volume for a site. Both of these practices may be used for pretreatment or included in a "treatment train" approach where redundant treatment is provided. Please see a further discussion of these limited application structural controls in subsections 5.2.12 and 5.2.3, respectively.

5.2.2.2 Storm Water Management Suitability

Enhanced swale systems are designed primarily for storm water quality and have only a limited ability to provide streambank protection or to convey higher flows to other controls.

Water Quality

Dry swale systems rely primarily on filtration through an engineered media to provide removal of storm water contaminants. Wet swales achieve pollutant removal both from sediment accumulation and biological removal.

Section 5.2.2.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

Generally, only the WQ_v is treated by a dry or wet swale, and another structural control must be used to provide SP_v extended detention. However, for some smaller sites, a swale may be designed to capture and detain the full SP_v.

On-Site Flood Control

Enhanced swales must provide flow diversion and/or be designed to safely pass overbank flood flows. Another structural control must be used in conjunction with an enhanced swale system to reduce the post-development peak flow.

Downstream Flood Control

Enhanced swales must provide flow diversion and/or be designed to safely pass extreme storm flows. Another structural control must be used in conjunction with an enhanced swale system to reduce the post-development peak flow.

5.2.2.3 Pollutant Removal Capabilities

Both the dry and wet enhanced swale are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed swales can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – Dry Swale 50% / Wet Swale 25%**
- **Total Nitrogen – Dry Swale 50% / Wet Swale 40%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – Dry Swale 40% / Wet Swale 20%**

For additional information and data on pollutant removal capabilities for enhanced dry and wet swales, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

5.2.2.4 Application and Feasibility Criteria

Enhanced swales can be used in a variety of development types; however, they are primarily applicable to residential and institutional areas of low to moderate density where the impervious cover in the contributing drainage area is relatively small and along roads and highways. Dry swales are mainly used in moderate to large lot residential developments, small impervious areas (parking lots and rooftops), and along rural highways. Wet swales tend to be used for highway runoff applications, small parking areas, and in commercial developments as part of a landscaped area.

Because of their relatively large land requirement, enhanced swales are generally not used in higher density areas. In addition, wet swales may not be desirable for some residential applications, due to the presence of standing and stagnant water, which may create nuisance odor or mosquito problems.

The topography and soils of a site will determine the applicability of the use of one of the two enhanced swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross-sectional area to maintain nonerosive velocities. The following criteria should be evaluated to ensure the suitability of a storm water pond for meeting storm water management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – NO
- Regional Storm Water Control – NO

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum
- Space Required – Approximately 10 to 20% of the tributary impervious area
- Site Slope – Typically no more than 4% channel slope
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 feet for dry swale; 1 foot for wet swale
- Minimum Depth to Water Table – 2 feet required between the bottom of a dry swale and the elevation of the seasonally high water table if treating a hotspot or an aquifer recharge zone. Wet swale is below water table or placed in poorly drained soils
- Soils – Engineered media for dry swale

Other Constraints / Considerations

- Aquifer Protection – Infiltration should not be allowed for hotspots

5.2.2.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of an enhanced swale system. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- A dry or wet swale should be sited such that the topography allows for the design of a channel with sufficiently mild slope (unless small drop structures are used) and cross-sectional area to maintain nonerosive velocities.
- Enhanced swale systems should have a contributing drainage area of 5 acres or less.

- Swale siting should also take into account the location and use of other site features, such as buffers and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape.
- A wet swale can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community.

B. GENERAL DESIGN

- Both types of enhanced swales are designed to treat the WQ_v through a volume-based design, and to safely pass larger storm flows. Flow enters the channel through a pretreatment forebay. Runoff can also enter along the sides of the channel as sheet flow through the use of a pea gravel flow spreader trench along the top of the bank.

Dry Swale

- A dry swale system consists of an open conveyance channel with a filter bed of permeable soils that overlays an underdrain system. Flow passes into and is detained in the main portion of the channel where it is filtered through the soil bed. Runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. Figure 5.2.2-2 provides a plan view and cross-section schematic for the design of a dry swale system.

Wet Swale

- A wet swale or wetland channel consists of an open conveyance channel which has been excavated to the water table or to poorly drained soils. Check dams are used to create multiple wetland "cells," which act as miniature shallow marshes. Figure 5.2.2-3 provides a plan view and cross-section schematic for the design of a wet swale system.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- Channel slopes between 1% and 2% are recommended unless topography necessitates a steeper slope, in which case 6- to 12-inch drop structures can be placed to limit the energy slope to within the recommended 1 to 2% range. Energy dissipation will be required below the drops. Spacing between the drops should not be closer than 50 feet. Depth of the WQ_v at the downstream end should not exceed 18 inches.
- Dry and wet swales should have a bottom width of 2 to 8 feet to ensure adequate filtration. Wider channels can be designed, but should contain berms, walls, or a multi-level cross section to prevent channel braiding or uncontrolled sub-channel formation.
- Dry and wet swales are parabolic or trapezoidal in cross section and are typically designed with moderate side slopes no greater than 2:1 for ease of maintenance and side inflow by sheet flow (4:1 or flatter recommended).
- Dry and wet swales should maintain a maximum WQ_v ponding depth of 18 inches at the end point of the channel. A 12-inch average depth should be maintained.
- The peak velocity for the 3 storm events ("Streambank Protection", "Conveyance", and 100-year) must be nonerosive for the soil and vegetative cover provided.
- If the system is on-line, channels should be sized to convey runoff from a flood event safely with a minimum freeboard and without damage to adjacent property.

Dry Swale

- Dry swale channels are sized to store and infiltrate the entire water quality volume (WQ_v) with less than 18 inches of ponding and allow for full filtering through the permeable soil layer. The maximum ponding time is 48 hours, though a 24-hour ponding time is more desirable.
- The bed of the dry swale consists of a permeable soil layer of at least 30 inches in depth, above a 4-inch diameter perforated PVC pipe (AASHTO M 252) longitudinal underdrain in a 6-inch gravel layer.

The soil media should have an infiltration rate of at least 1 foot per day (1.5 feet per day maximum) and contain a high level of organic material to facilitate pollutant removal. A permeable filter fabric is placed between the gravel layer and the overlying soil.

- The channel and underdrain excavation should be limited to the width and depth specified in the design. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction and scarified prior to placement of gravel and permeable soil. The sides of the channel shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling.

Wet Swale

- Wet swale channels are sized to retain the entire water quality volume (WQ_v) with less than 18 inches of ponding at the maximum depth point.
- Check dams can be used to achieve multiple wetland cells. V-notch weirs in the check dams can be utilized to direct low flow volumes.

D. PRETREATMENT / INLETS

- Inlets to enhanced swales must be provided with energy dissipaters such as riprap.
- Pretreatment of runoff in both a dry and wet swale system is typically provided by a sediment forebay located at the inlet. The pretreatment volume should be equal to 0.1 inches per impervious acre. This storage is usually obtained by providing check dams at pipe inlets and/or driveway crossings.
- Enhanced swale systems that receive direct concentrated runoff may have a 6-inch drop to a pea gravel diaphragm flow spreader at the upstream end of the control.
- A pea gravel diaphragm and gentle side slopes should be provided along the top of channels to provide pretreatment for lateral sheet flows.

E. OUTLET STRUCTURES

Dry Swale

- The underdrain system should discharge to the storm drainage infrastructure or a stable outfall.

Wet Swale

- Outlet protection must be used at any discharge point from a wet swale to prevent scour and downstream erosion.

F. EMERGENCY SPILLWAY

- Enhanced swales must be adequately designed to safely pass flows that exceed the design storm flows.

G. MAINTENANCE ACCESS

- Adequate access should be provided for all dry and wet swale systems for inspection and maintenance.

H. SAFETY FEATURES

- Ponding depths should be limited to a maximum of 18 inches.

I. LANDSCAPING

Landscape design should specify proper grass species and wetland plants based on specific site, soils, and hydric conditions present along the channel. Below is some specific guidance for dry and wet swales:

Dry Swale

- Information on appropriate turf grass species for North Central Texas can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.

Wet Swale

- Emergent vegetation should be planted, or wetland soils may be spread on the swale bottom for seed stock.
- Information on establishing wetland vegetation and appropriate wetland species for North Central Texas can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.
- Where wet swales do not intercept the groundwater table, a water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. See subsection 2.1.11 for guidance on water balance calculations.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Reduced need for use of check dams
- High Relief – Often infeasible if slopes are greater than 4%
- Karst – No infiltration of hotspot runoff from dry swales; use impermeable liner

Soils

No additional criteria

Special Downstream Watershed Considerations

- Aquifer Protection – No infiltration of hotspot runoff from dry swales; use impermeable liner

5.2.2.6 Design Procedures

Step 1. Compute runoff control volumes from the *integrated* Design Approach

Calculate the Water Quality Volume (WQ_v), Streambank Protection Volume (SP_v), On-Site Flood Control Volume (V_s), and the Downstream Flood Control Volume (V_f).

Details on the *integrated* Design Approach are found in Section 1.2.

Step 2. Determine if the development site and conditions are appropriate for the use of an enhanced swale system (dry or wet swale).

Consider the Application and Site Feasibility Criteria in subsections 5.2.2.4 and 5.2.2.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 5.2.2.5-J (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Determine pretreatment volume

The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage. The forebay storage volume counts toward the total WQ_v requirement, and should be subtracted from the WQ_v for subsequent calculations.

Step 5. Determine swale dimensions

Size bottom width, depth, length, and slope necessary to store WQ_v with less than 18 inches of ponding at the downstream end.

- Slope cannot exceed 4% (1 to 2% recommended)
- Bottom width should range from 2 to 8 feet
- Ensure that side slopes are no greater than 2:1 (4:1 recommended)

See subsection 5.2.2.5-C (Physical Specifications / Geometry) for more details

Step 6. Compute number of check dams (or similar structures) required to detain WQ_v

Step 7. Calculate draw-down time

Dry swale: Planting soil should pass a maximum rate of 1.5 feet in 24 hours and must completely filter WQ_v within 48 hours.

Wet swale: Must hold the WQ_v.

Step 8. Check low flow and design event velocity erosion potential and freeboard

Check for erosive velocities and modify design as appropriate. Provide 6 inches of freeboard.

Step 9. Design low flow orifice at downstream headwalls and checkdams

Design orifice to pass WQ_v in 6 hours. Use Orifice equation.

Step 10. Design inlets, sediment forebay(s), and underdrain system (dry swale)

See subsection 5.2.2.5-D through H for more details.

Step 11. Prepare Vegetation and Landscaping Plan

A landscaping plan for a dry or wet swale should be prepared to indicate how the enhanced swale system will be stabilized and established with vegetation.

See subsection 5.2.2.5-I (Landscaping) and Appendix F for more details.

See Appendix D-5 for an Enhanced Swale Design Example

5.2.2.7 Inspection and Maintenance Requirements

Table 5.2.2-1 Typical Maintenance Activities for Enhanced Swales
(Source: WMI, 1997; Pitt, 1997)

Activity	Schedule
• For dry swales, mow grass to maintain a height of 4 to 6 inches. Remove grass clippings.	As needed (frequent/seasonally)
• Inspect grass along side slopes for erosion and formation of rills or gullies and correct. • Remove trash and debris accumulated in the inflow forebay. • Inspect and correct erosion problems in the sand/soil bed of dry swales. • Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established. • Replant wetland species (for wet swale) if not sufficiently established. • Inspect pea gravel diaphragm for clogging and correct the problem.	Annually (Semi-annually the first year)
• Roto-till or cultivate the surface of the sand/soil bed of dry swales if the swale does not draw down within 48 hours. • Remove sediment build-up within the bottom of the swale once it has accumulated to 25% of the original design volume.	As needed

Regular inspection and maintenance is critical to the effective operation of an enhanced swale system as designed. Maintenance responsibility for a dry or wet swale should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

5.2.2.8 Example Schematics

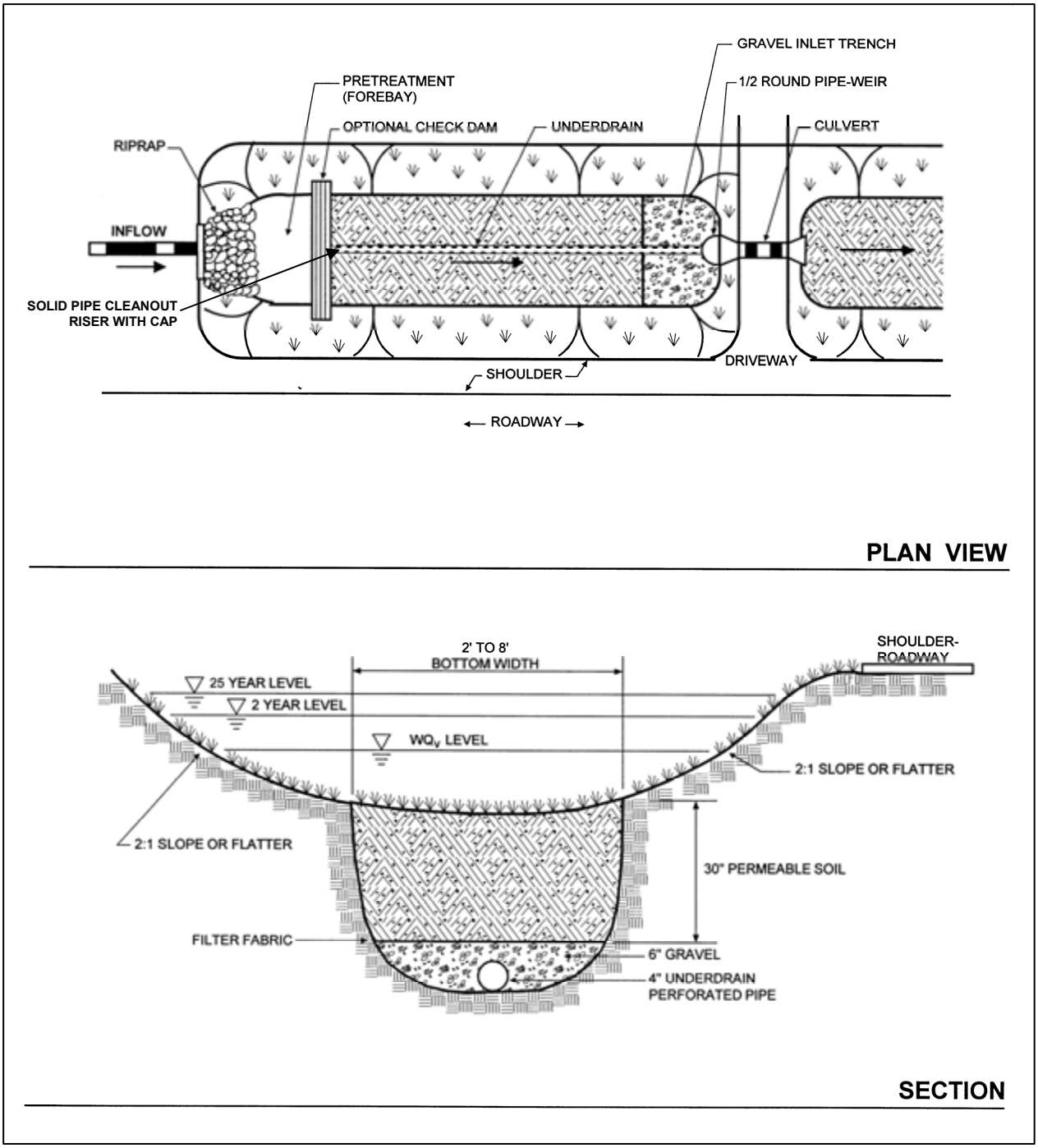


Figure 5.2.2-2 Schematic of Dry Swale
(Source: Center for Watershed Protection)

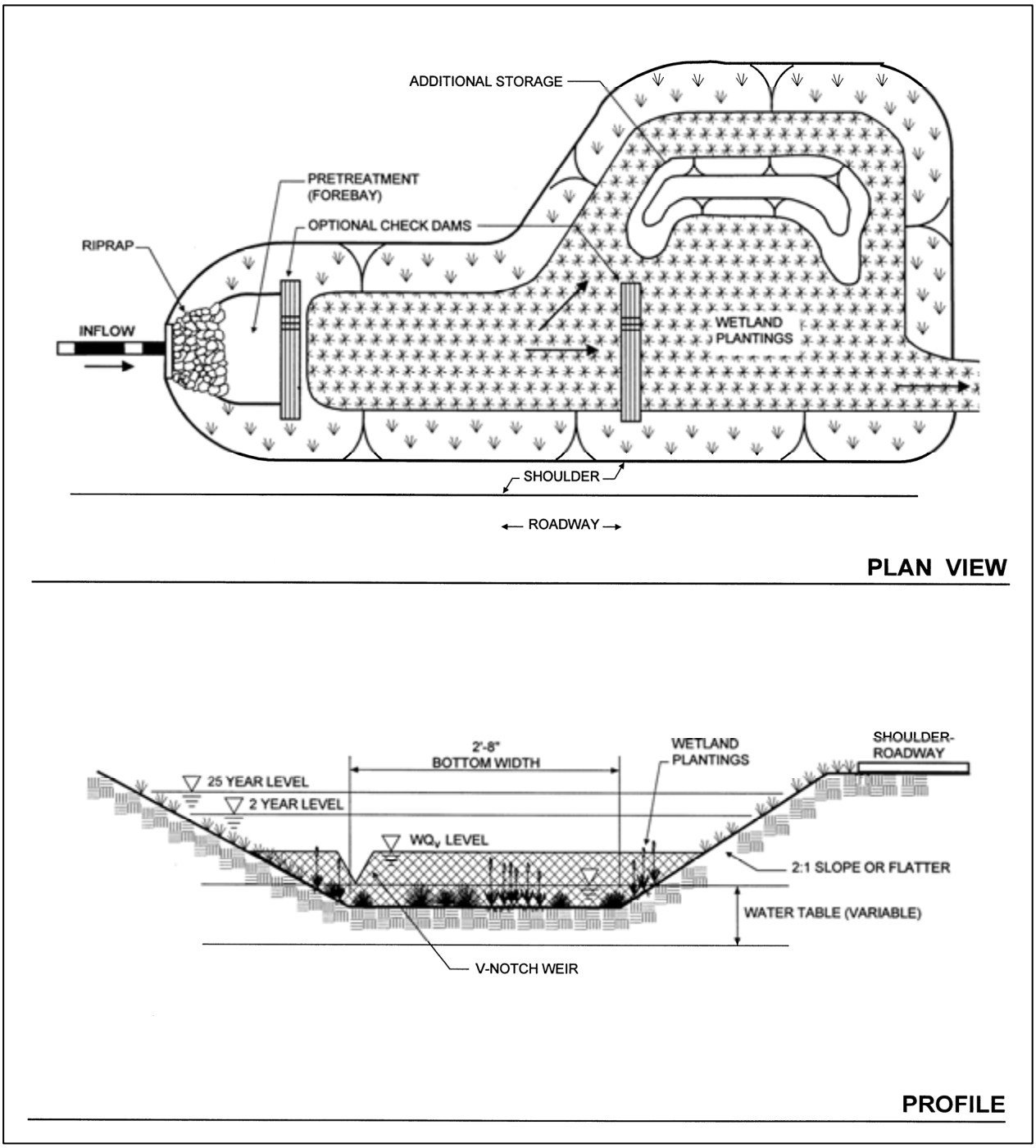


Figure 5.2.2-3 Schematic of Wet Swale
(Source: Center for Watershed Protection)

5.2.2.9 Design Forms

Design Procedure Form: Enhanced Swales

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements Compute Runoff Coefficient, R_v Compute WQ_v	R_v = _____ WQ_v = _____ acre-ft
1b. Compute SP_v Compute average release rate Compute Q_p (100-year detention volume required) Compute (as necessary) Q_f	SP_v = _____ acre-ft release rate = _____ cfs Q_p = _____ acre-ft Q_f = _____ cfs
ENHANCED SWALE DESIGN	
2. Is the use of an enhanced swale appropriate?	***See subsections 5.2.2.4 and 5.2.2.5 - A***
3. Confirm local design criteria and applicability.	***See subsection 5.2.2.5 - J***
4. Pretreatment Volume Vol_{pre} = I (0.1")(1'/12")	Vol_{pre} = _____ acre-ft
5. Determine swale dimensions Assume trapezoidal channel with max depth of 18 inches	Length = _____ ft Width = _____ ft Side Slopes = _____ Area = _____ ft^2
6. Compute number of check dams (or similar structures) required to detain WQ_v	Slope = _____ ft/ft Depth = _____ ft Distance = _____ ft Number = _____ each
7. Calculate draw-down time Require k = 1.5 ft per day for dry swales	t = _____ hr
8. Check low flow and design storm velocity erosion potential and freeboard Requires separate computer analysis for velocity	V_{min} = _____ fps
Overflow wier (use weir equation) Use weir equation for slot length ($Q = CLH^{3/2}$)	Weir Length = _____ ft
9. Design low flow orifice at headwall Area of orifice from orifice equation $Q = CA(2gh)^{0.5}$	Area = _____ ft^2 diam = _____ inch
10. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.	***See subsection 5.2.2.5 - D through H***
11. Attach landscaping plan (including wetland vegetation)	***See Appendix F***

Notes: _____

5.2.3 Grass Channel

Structural Storm Water Control

Description: Vegetated open channels designed to filter storm water runoff and meet velocity targets for the water quality design storm and the "Streambank Protection" storm event.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Should not be used on slopes greater than 4%; slopes between 1% and 2% recommended
- Ineffective unless carefully designed to achieve low flow rates in the channel (<1.0 ft/s)

ADVANTAGES / BENEFITS:

- Can be used as part of the runoff conveyance system to provide pretreatment
- Grass channels can act to partially infiltrate runoff from small storm events if underlying soils are pervious
- Less expensive to construct than curb and gutter systems

DISADVANTAGES / LIMITATIONS:

- May require more maintenance than curb and gutter system
- Cannot alone achieve the 80% TSS removal target
- Potential for bottom erosion and re-suspension
- Standing water may not be acceptable in some areas

POLLUTANT REMOVAL

50%	**Total Suspended Solids**
25/20%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
30%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

S	**Water Quality Protection**
S	**Streambank Protection**
P	**On-Site Flood Control**
S	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

H	**Land Requirement**
L	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Curb and gutter replacement*

L=Low **M**=Moderate **H**=High

5.2.3.1 General Description

Grass channels, also termed "biofilters," are typically designed to provide nominal treatment of runoff as well as meet runoff velocity targets for the water quality design storm. Grass channels are well suited to a number of applications and land uses, including treating runoff from roads and highways and pervious surfaces.

Grass channels differ from the enhanced dry swale design in that they do not have an engineered filter media to enhance pollutant removal capabilities and, therefore, have a lower pollutant removal rate than for a dry or wet (enhanced) swale. Grass channels can partially infiltrate runoff from small storm events in areas with pervious soils. When properly incorporated into an overall site design, grass channels can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

When designing a grass channel, the two primary considerations are channel capacity and minimization of erosion. Runoff velocity should not exceed 1.0 foot per second during the peak discharge associated with the water quality design rainfall event, water depth should generally be less than 4 inches (height of the grass), and the total length of a grass channel should provide at least 5 minutes of residence time. To enhance water quality treatment, grass channels must have broader bottoms, lower slopes, and denser vegetation than most drainage channels. Additional treatment can be provided by placing check-dams across the channel below pipe inflows, and at various other points along the channel.

5.2.3.2 Pollutant Removal Capabilities

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- **Total Suspended Solids – 50%**
- **Total Phosphorus – 25%**
- **Total Nitrogen – 20%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 30%**

Fecal coliform removal is uncertain. In fact, grass channels are often a source of fecal coliforms from local residents walking their dogs.

5.2.3.3 Design Criteria and Specifications

- Grass channels should generally be used to treat small drainage areas of less than 5 acres. If the practices are used on larger drainage areas, the flows and volumes through the channel become too large to allow for filtering and infiltration of runoff.
- Grass channels should be designed on relatively flat slopes of less than 4%; channel slopes between 1% and 2% are recommended.
- Grass channels can be used on most soils with some restrictions on the most impermeable soils. Grass channels should not be used on soils with infiltration rates less than 0.27 inches per hour if infiltration of small runoff flows is intended.
- A grass channel should accommodate the peak flow for the water quality design storm Q_{wq} (see Section 2.1.10).
- Grass channels should have a trapezoidal or parabolic cross section with relatively flat side slopes (generally 3:1 or flatter).
- The bottom of the channel should be between 2 and 6 feet wide. The minimum width ensures an adequate filtering surface for water quality treatment, and the maximum width prevents braiding, which is the formation of small channels within the swale bottom. The bottom width is a dependent

variable in the calculation of velocity based on Manning's equation. If a larger channel is needed, the use of a compound cross section is recommended.

- Runoff velocities must be nonerosive. The full-channel design velocity will typically govern.
- A 5-minute residence time is recommended for the water quality peak flow. Residence time may be increased by reducing the slope of the channel, increasing the wetted perimeter, or planting a denser grass (raising the Manning's n).
- The depth from the bottom of the channel to the groundwater should be at least 2 feet to prevent a moist swale bottom, or contamination of the groundwater.
- Incorporation of check dams within the channel will maximize retention time.
- Designers should choose a grass that can withstand relatively high velocity flows at the entrances for both wet and dry periods. See Appendix F for a list of appropriate grasses for use in North Central Texas.

See Section 4.4 (*Open Channel Design*) for more information and specifications on the design of grass channels.

Grass Channels for Pretreatment

A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a grass channel as a pretreatment measure. The length of the grass channel depends on the drainage area, land use, and channel slope. Table 5.2.3-1 provides sizing guidance for grass channels for a 1-acre drainage area. The minimum grassed channel length should be 20 feet.

Table 5.2.3-1 Bioretention Grass Channel Sizing Guidance

Parameter	<= 33% Impervious		Between 34% and 66% Impervious		>= 67% Impervious	
Slope (max = 4%)	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%
Grass channel minimum length* (feet) *assumes 2-foot wide bottom width	25	40	30	45	35	50

(Source: Claytor and Schueler, 1996)

5.2.3.4 Inspection and Maintenance Requirements

Table 5.2.3-2 Typical Maintenance Activities for Grass Channels

Activity	Schedule
• Mow grass to maintain a height of 3 to 4 inches.	As needed (frequently/seasonally)
• Remove sediment build-up within the bottom of the grass channel once it has accumulated to 25% of the original design volume.	As needed (Infrequently)
• Inspect grass along side slopes for erosion and formation of rills or gullies and correct. • Remove trash and debris accumulated in the channel. • Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established.	Annually (Semi-annually the first year)

(Source: Adapted from CWP, 1996)

5.2.3.5 Example Schematics

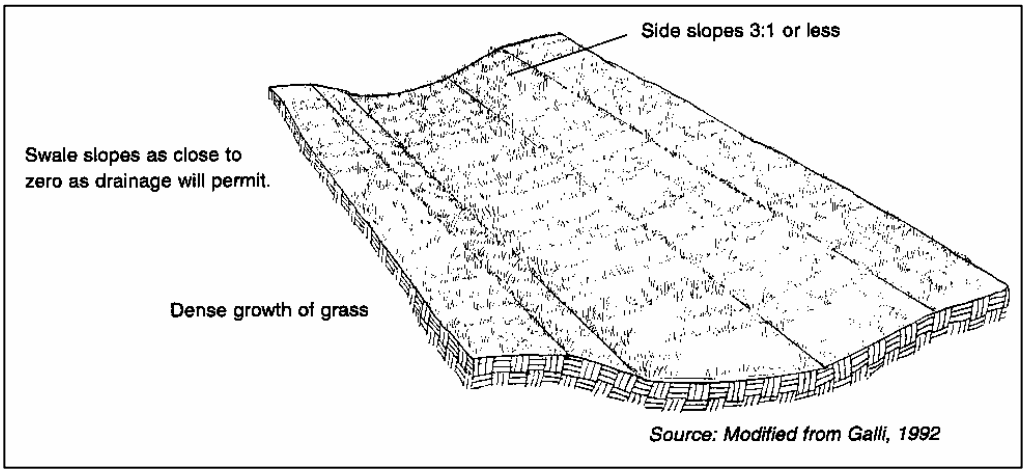


Figure 5.2.3-1 Typical Grass Channel

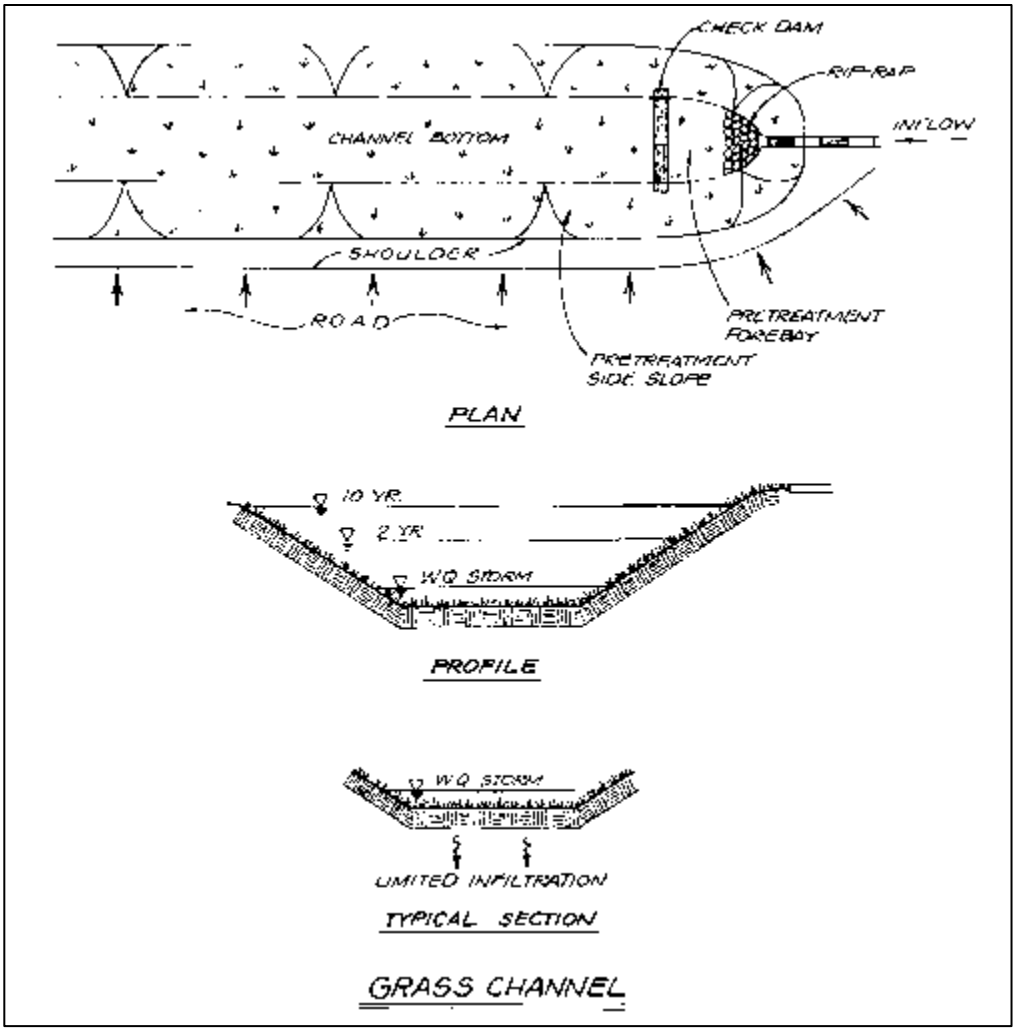


Figure 5.2.3-2 Schematic of Grass Channel

5.2.3.6 Design Example

Basic Data

Small commercial lot 300 feet deep x 145 feet wide

- Drainage area (A) = 1.0 acres
- Impervious percentage (I) = 70%

Water Quality Peak Flow

See subsection 1.4.2.1 for details

Compute the Water Quality Protection Volume in inches:

$WQ_v = 1.5\ (0.05 + 0.009 * 70) = 1.02$ inches

Compute modified CN for 1.5-inch rainfall (P=1.5):

$$CN = 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{1/2}]$$
$$= 1000/[10+5*1.5+10*0.82-10(0.82^2+1.25*0.82*1.5)^{1/2}]$$
$$= 92.4 \quad \text{(Use CN = 92)}$$

For CN = 92 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.5-inch storm.

From Table 2.1.5-3, I_a = 0.174, therefore I_a/P = 0.174/1.5 = 0.116.

From Figure 2.1.5-6 for a Type II storm (using the limiting values) q_u = 950 csm/in, and therefore:

Q_{wq} = (950 csm/in) (1.0ac/640ac/mi^2) (1.02") = 1.51 cfs

Utilize Q_{wq} to Size the Channel

The maximum flow depth for water quality treatment should be approximately the same height of the grass. A maximum flow depth of 4 inches is allowed for water quality design. A maximum flow velocity of 1.0 foot per second for water quality treatment is required. For Manning's n use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass. Site slope is 2%.

Input variables:

n = 0.15
S = 0.02 ft/ft
D = 4/12 = 0.33 ft

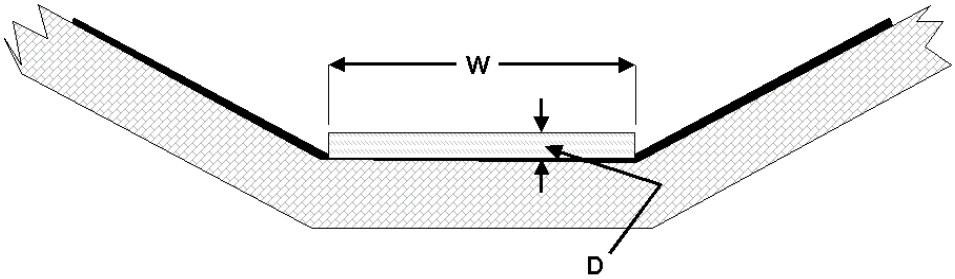
Then:

$Q_{wq} = Q = VA = 1.49/n\ D^{2/3}\ S^{1/2}\ DW$

where:

Q = peak flow (cfs)
V = velocity (ft/sec)
A = flow area (ft^2) = WD
W = channel bottom width (ft)
D = flow depth (ft)
S = slope (ft/ft)

(Note: D approximates hydraulic radius for shallow flows)



Then for a known n, Q, D and S minimum width can be calculated.

$(nQ)/(1.49\ D^{5/3}\ S^{1/2}) = W = (0.15*1.51)/(1.49*0.33^{5/3}*0.02^{1/2}) = 6.84$ feet minimum

$V = Q/(WD) = 1.51/(6.84 * 4/12) = 0.66$ fps (okay)

(Note: WD approximates flow area for shallow flows.)

Minimum length for 5-minute residence time, $L = V * (5*60) = 198$ feet

Depending on the site geometry, the width or slope or density of grass (Manning's n value) might be adjusted to slow the velocity and shorten the channel in the next design iteration. For example, using a 10-foot bottom width* of flow and a Manning's n of 0.20, solve for new depth and length.

$Q = VA = 1.49/n\ D^{5/3}\ S^{1/2}\ W$

$D = [(Q * n)/(1.49 * S^{1/2} * W)]^{3/5}$

$= [(1.51 * 0.20)/(1.49 * 0.02^{1/2} * 10.0)]^{3/5} = 0.31$ ft = 4" (okay)

$V = Q/WD = 1.51/(10.0 * 0.31) = 0.49$ feet per second

$L = V * 5 * 60 = 146$ feet

** In this case a dividing berm should be used to control potential braiding.*

Refer to Section 4.4 (*Open Channel Design*) to complete the grass channel design for a specified design storm event.

5.2.4 Open Conveyance Channel

Storm Water Control

Description: An open channel is a conduit in which water flows with a free surface. Open channels include conveyance channels or drainage ditches; grass channels; and enhanced swales. An open conveyance channel is designed for conveyance purposes only.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- A maximum of 2:1 should be used for channel side slopes. Roadside ditches should have maximum side slope of 3:1.
- Slope stability should be confirmed with a geotechnical study or investigation
- Channel banks should be stabilized at site

ADVANTAGES / BENEFITS:

- Can be aesthetically pleasing
- Vegetated channels provide natural habitats
- Once established, little maintenance is required

DISADVANTAGES / LIMITATIONS:

- Velocity will limit the type of channel lining, for example, vegetated channels require slower velocities and lower longitudinal slopes

STORM WATER MANAGEMENT SUITABILITY

	Water Quality Protection
	Streambank Protection
P	**On-Site Flood Control**
S	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

H	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.4.1 General Description

An open channel is a conduit in which water flows with a free surface. Open channel systems and their design are an integral part of storm water drainage design, particularly for development sites utilizing better sited design practices and open channel structural controls. The broad category of open channels includes conveyance channels or drainage ditches, grass channels, and dry and wet enhanced swales. Grass channels and enhanced swales are designed to provide water quality benefits and are further described in detail in Sections 5.2.3 and 5.2.2, respectively.

Channel Classifications

Open channels may be classified into three main categories according to the type of channel linings: vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Flexible linings include stone riprap and some forms of flexible man-made linings or gabions. Rigid linings are generally concrete or rigid block.

Vegetated Lining Flexible Lining Rigid Lining

Figure 5.2.4-1 Open Channel Examples

5.2.4.2 Pollutant Removal Capabilities

Open conveyance channels or drainage ditches are designed for conveyance purposes only. For open channels with pollutant capabilities, refer to Sections 5.2.3 and 5.2.2, Grass Channels and Enhanced Swales, respectively.

5.2.4.3 Design Criteria and Specifications

Detailed design criteria and specifications, as prepared by the Federal Highway Administration, are presented in Section 4.4, Open Channel Design. Uniform flow, critical flow, and design details for the three main categories of channel classification (vegetative, riprap, and rigid lining) are also included in the noted Section.

In general, the following criteria should be followed for open channel design:

- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
- Channel side slopes shall be stable throughout the entire length and side slope shall depend on the channel material. A maximum of 2:1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum side slope of 3:1. All side slopes should be verified with a geotechnical evaluation to ensure slope stability.
- Trapezoidal or parabolic cross sections are preferred over triangular shapes.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

- Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.
- Open channel drainage systems are sized to adequately convey the “Conveyance” design storm, and are normally checked with the 100-year storm event.

5.2.4.4 Inspection and Maintenance Requirements

Open channels should be inspected after large storm events for debris causing blockages or re-routing. Channels with vegetated linings should be inspected and maintained periodically to insure vegetation is still in place and prevent growth of taller or woody vegetation. Flexible linings, such as rock riprap, have self-healing qualities that reduce maintenance. However, they should be inspected and maintained periodically to prevent growth of trees, grass, and weeds. Concrete channels should be checked periodically for scour at the channel lining transitions and channel headcutting.

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5.2.5 Alum Treatment System

Structural Storm Water Control

Description: Chemical treatment of storm water runoff entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events.

KEY CONSIDERATIONS

ADVANTAGES / BENEFITS:

- Requires no additional land purchase
- Reduces concentrations of total phosphorus, total aluminum and heavy metals
- Dependent on pH level ranging from 6.0 to 7.5 during treatment process

DISADVANTAGES / LIMITATIONS:

- Intended for areas requiring regional storm water treatment from a piped storm water drainage system
- High maintenance requirements
- Alum application will lower pH of receiving waters
- High capital and operations and maintenance costs

POLLUTANT REMOVAL

90%	**Total Suspended Solids**
80/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
75%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [L] **Land Requirement**
- [H] **Capital Cost**
- [H] **Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *25 acres min.*

Soils: *No restrictions*

Other Considerations:

- *Regional Treatment*

L=Low **M**=Moderate **H**=High

5.2.5.1 General Description

The process of alum (aluminum sulfate) treatment provides treatment of storm water runoff from a piped storm water drainage system entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. When added to runoff, liquid alum forms nontoxic precipitates of aluminum hydroxide [$Al(OH)_3$] and aluminum phosphate [$AlPO_4$]. However, Alum will lower the pH of receiving waters and must be closely monitored to avoid adverse impacts on aquatic life.

The alum precipitate or "floc" formed during coagulation of storm water combines with phosphorus, suspended solids, and heavy metals and removes them from the water column. The floc can be allowed to settle in receiving water or collected in small settling basins. Once settled, the floc is stable in sediments and will not re-dissolve due to changes in redox potential or pH under conditions normally found in surface water bodies. Laboratory or field testing may be necessary to verify feasibility and to establish design, maintenance, and operational parameters, such as the optimum coagulant dose required to achieve the desired water quality goals, chemical pumping rates and pump sizes.

Construction costs for existing alum storm water treatment facilities in Florida have ranged from $135,000 to $400,000. The capital construction costs of alum storm water treatment systems is independent of watershed size and depends primarily on the number of outfall locations treated.

Estimated annual operations and maintenance (O&M) costs for chemicals and routine inspections range from approximately $6,500 to $25,000 per year. O&M costs include chemical, power, manpower for routine inspections, equipment renewal, and replacement costs.

Ferric chloride has also been used for flow-proportional injection for removing phosphorus and other pollutants. Although ferric chloride is less toxic to aquatic life than alum, it has a number of significant disadvantages. Ferric chloride dosage rates are dependent on the pollutant concentrations in the storm water runoff, unlike alum. Ferric chloride does not form a floc that settles out suspended pollutants. And, once settled, ferric chloride may be released from sediments under anoxic conditions.

5.2.5.2 Pollutant Removal Capabilities

Alum treatment has consistently achieved a 85 to 95% reduction in total phosphorus, 90 to 95% reduction in orthophosphorus, 60 to 70% reduction in total nitrogen, 50 to 90% reduction in heavy metals, 95 to 99% reduction in turbidity and TSS, 60% reduction in BOD, and >99% reduction in fecal coliform bacteria compared with raw storm water characteristics.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- **Total Suspended Solids – 90%**
- **Total Phosphorus – 80%**
- **Total Nitrogen – 60%**
- **Fecal Coliform – 90%**
- **Heavy Metals – 75%**

5.2.5.3 Design Criteria and Specifications

Alum treatment systems are fairly complex, and design details are beyond the scope of this Manual. However, further information can be obtained from the Internet and by contacting local municipalities and engineers who have designed and implemented successful systems. The following are general guidelines for alum treatment systems:

- Injection points should be 100 feet upstream of discharge points.

- Alum concentration is typically 10 μg/l.
- Alum treatment systems may need to control pH.
- For new pond design, the required size is approximately 1% of the drainage basin size, as opposed to 10 to 15% of the drainage basin area for a standard detention pond.
- No additional volume is required when discharging to existing lakes.

5.2.5.4 Inspection and Maintenance Requirements

Table 5.2.5-1 Typical Maintenance Activities for Alum Treatment

Activity	Schedule
• Perform routine inspection. • Monitor water quality and pH of receiving water.	Monthly
• Perform maintenance of pump equipment, chemical supplies, and delivery system.	As Needed

(Source: Harper, Herr, and Livingston)

Figure 5.2.5-1. Alum Treatment System and Injection Equipment

5.2.6 Culvert

Storm Water Control

Description: A short, closed (covered) conduit that conveys storm water runoff under an embankment, usually a roadway.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for conveyance purposes, not pollutant removal capability
- Normally designed for the "Conveyance" storm event

VELOCITY:

- Maximum velocity of 15 fps for corrugated metal pipe
- Minimum velocity of 2.5 fps, for the "Streambank Protection" storm event

SLOPE:

- Maximum slope of 14% for corrugated metal pipe
- Maximum slope of 10% for concrete pipe
- Maximum drop in a drainage structure is 10 feet.

OTHER:

- Skew not to exceed 45 degrees
- Minimum diameter of 18 inches

MAINTENANCE REQUIREMENTS:

- Reinforced concrete pipe for use (1) under roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams.
- RCP and fully coated corrugated metal pipe
- High-density polyethylene (HDPE) may be used as specified in municipal regulations

STORM WATER MANAGEMENT SUITABILITY

	Water Quality Protection
	Streambank Protection
P	**On-Site Flood Control**
P	**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: *No restrictions.*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.6.1 General Description

A culvert is a short, closed (covered) conduit that conveys storm water runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows. In addition to the hydraulic function, a culvert must also support the embankment and/or roadway, and protect traffic and adjacent property owners from flood hazards to the extent practicable.

5.2.6.2 Pollutant Removal Capabilities

Culverts are designed for storm water conveyance purposes and do not provide pollutant removal capabilities.

5.2.6.3 Design Criteria and Specifications

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable:

- Frequency Flood;
- Velocity Limitations;
- Buoyancy Protection;
- Length and Slope;
- Debris Control;
- Headwater Limitations;
- Tailwater Considerations;
- Storage;
- Inlets;
- Inlets with Headwalls;
- Wingwalls and Aprons;
- Improved Inlets;
- Material Selection;
- Culvert Skews;
- Culvert Sizes;
- Weep Holes;
- Outlet Protection;
- Erosion and Sediment Control; and
- Environmental Considerations.

There are two types of flow conditions for culverts (see Figure 5.2.6-1) that are based upon the location of the control section and the critical flow depth:

- Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

- Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs.

There are three procedures for designing culverts: manual use of inlet and outlet control equations, nomographs, and the use of computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

Examples of small culverts are shown in Figure 5.2.6-2. See Section 4.2 for detailed culvert design procedures and instruction.

5.2.6.4 Inspection and Maintenance Requirements

Culverts located at the end of urban drainage channels are often clogged by refuse dumped into the channel or by trash washed off the city streets. Under such conditions, a debris rack can usually be installed at a low cost to prevent clogging. In designing debris control structures it is recommended that the Federal Highway Administration, Hydraulic Engineering Circular No. 9 entitled *Debris Control Structures* be consulted. This Circular discusses the variety of methods for controlling debris by: (a) intercepting the debris at or above the inlet; (b) deflecting the debris for detention near the inlet; or (c) passing the debris through the structure.

Additionally, to ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 feet per second, for the 2-year flow, when the culvert is flowing partially full is required.

5.2.6.5 Example Schematic

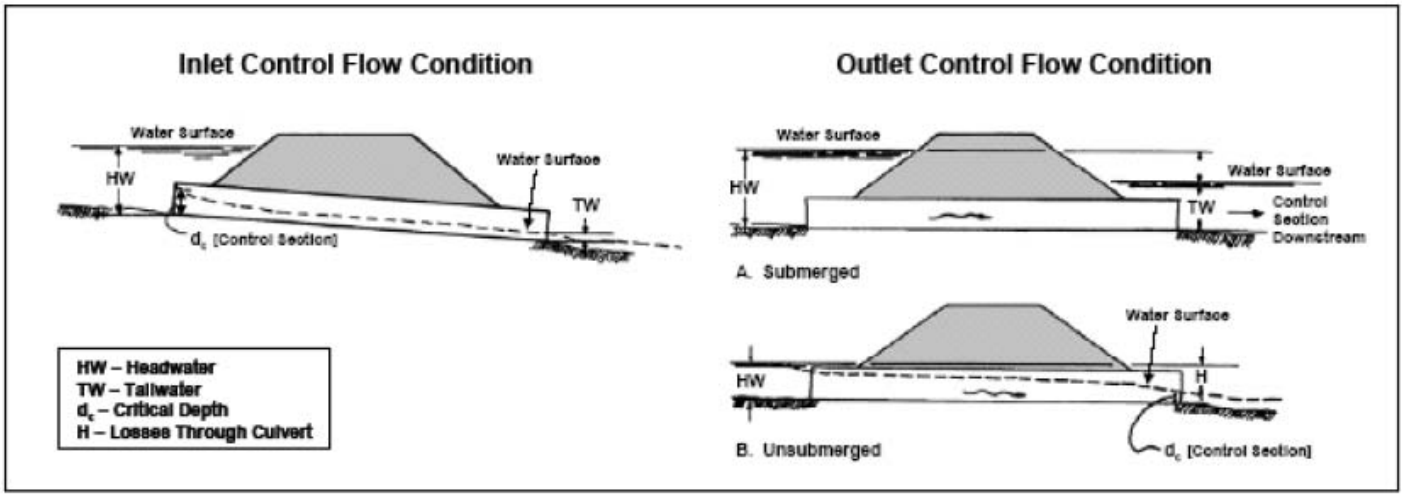

Figure 5.2.6-1 Culvert Flow Conditions

Figure 5.2.6-2 Culvert Examples

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5.2.7 Inlets

Storm Water Controls

Description: Drainage structure used to collect surface water through grate or curb openings and convey it to pipe systems or direct outlet to culverts.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for conveyance purposes, not pollutant removal capability
- Interception capacity depends on depth of water next to curb.

ADVANTAGES / BENEFITS:

- Aesthetically pleasing, less obvious than most storm water structures.

DISADVANTAGES / LIMITATIONS:

- Efficiency is typically reduced in order to meet safety standards.

MAINTENANCE REQUIREMENTS:

- Must be inspected and cleaned regularly due to trash and debris build-up.

STORM WATER MANAGEMENT SUITABILITY

- [ ] **Water Quality Protection**
- [ ] **Streambank Protection**
- [P] **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: *No restrictions.*

Soils: *No restrictions*

Other Considerations:

- *Overflow parking, driveways and related issues*

L=Low **M**=Moderate **H**=High

5.2.7.1 General Description

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to pipe systems or direct outlet to culverts. Inlets are typically located in close proximity to impervious areas such as streets and parking lots. Inlets can also be located at a low point in a channel or area that concentrates overland flow (ponding areas).

5.2.7.2 Pollutant Removal Capabilities

Although inlets prevent large debris from passing to the storm sewer system, they are designed for storm water conveyance purposes and do not provide pollutant removal capabilities.

5.2.7.3 Design Criteria and Specifications

Inlets used for drainage of surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets, consisting of an opening in the gutter covered by one or more grates, and slotted inlets, consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as on a *continuous grade* or in a *sump*. The term “continuous grade” refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The “sump” condition exists when the inlet is located at a low point and water enters from both directions. There are specific design criteria for the following types of inlets:

- Grate Inlets on Grade
- Grate Inlets in Sag
- Curb Inlets on Grade
- Curb Inlets in Sump
- Combination Inlets on Grade
- Combination Inlets in Sump

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

When designing an inlet, the grate length, bar configuration, and gutter velocity must be taken into account. Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, and should take advantage of natural depression storage where possible. Grate inlets subject to traffic should be bicycle safe (horizontal and vertical cross-bars) and provide adequate load-bearing capabilities.

See Section 4.2 for detailed inlet design procedures and instruction.

5.2.7.4 Inspection and Maintenance Requirements

Inlets are often blocked by trash washed off the city streets and parking lots. Regular inspection and removal of this debris will result in cleaner, more efficient storm water conveyance systems. The public

might also misunderstand inlets as places to dispose of household wastes. To prevent this and to promote water quality education, some cities and non-profit organizations have begun “stamping” inlets or attaching decals with “No Dumping” instructions.

Grate Inlet in Parking Lot

Slotted Inlet

Curb Inlet

Stamp on Inlet

Combination Inlet

Figure 5.2.7-1 Example Inlets

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5.2.8 Pipe Systems

Storm Water Control

Description: Pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at structural storm water controls and receiving waters.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for conveyance purposes, not pollutant removal capability
- All systems should be designed with velocities greater than 2.5 fps with a minimum slope of 0.5%
- Size system under the assumption of full flow, but not pressure flow
- Hydraulic gradient should not produce velocity in excess of 15 fps
- Manning's Equation recommended for capacity computations

STORM WATER MANAGEMENT SUITABILITY

	Water Quality Protection
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**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: *No restrictions.*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.8.1 General Description

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used in the minor storm water drainage system for transporting runoff from roadway and other inlets to outfalls at structural storm water controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

5.2.8.2 Pollutant Removal Capabilities

The storm water pipe system is designed for conveyance purposes and does not provide pollutant removal capabilities.

5.2.8.3 Design Criteria and Specifications

The design of storm drain systems generally follows these steps:

Step 1 Determine inlet location and spacing.

Step 2 Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

Step 3 Determine drainage areas and compute runoff using the Rational Method

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (Figure 3.2-12) can be used to summarize hydrologic, hydraulic, and design computations.

Step 5 Examine assumptions to determine if any adjustments are needed to the final design.

It should be recognized that the rate of discharge to be carried by any particular section of storm drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

See Section 3.2.8 for detailed pipe system design procedures and instruction.

5.2.8.4 Inspection and Maintenance Requirements

Maintaining storm water conveyance structures on a regular basis will prevent clogging of the downstream conveyance system and ensure the system functions properly hydraulically to avoid flooding.

Storm Drain Conveyance System

- Locate reaches of storm drain with deposit problems and develop a flushing schedule that keeps the pipe clear of excessive buildup.
- Collect flushed effluent and pump to the sanitary sewer for treatment after sediment removal, if necessary.

Illicit Connections and Discharges

- During routine maintenance of conveyance system and drainage structures field staff should look for evidence of illegal discharges or illicit connections:
  - Is there evidence of spills such as paints, discoloring, etc.

  - Are there any odors associated with the drainage system
  - Record locations of apparent illegal discharges/illicit connections
  - Track flows back to potential dischargers and conduct aboveground inspections. This can be done through visual inspection of up gradient manholes or alternate techniques including zinc chloride smoke testing, fluorometric dye testing, physical inspection testing, or television camera inspection.
  - Once the origin of flow is established, require illicit discharger to eliminate the discharge.
- Stencil storm drains, where applicable, to prevent illegal disposal of pollutants. Storm drain inlets should have messages such as "Dump No Waste Drains to Stream" stenciled next to them to warn against uninformed or intentional dumping of pollutants into the storm drainage system.

Illegal Dumping

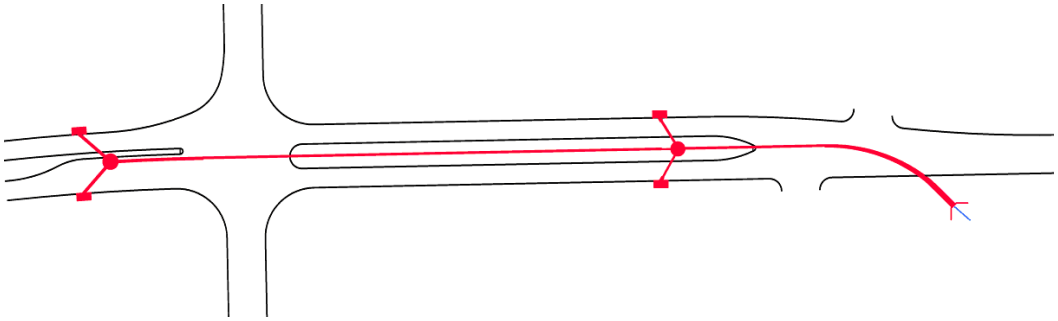
- Regularly inspect and clean up hot spots and other storm drainage areas where illegal dumping and disposal occurs.
- Establish a system for tracking incidents. The system should be designed to identify the following:
  - Illegal dumping hot spots
  - Types and quantities (in some cases) of wastes
  - Patterns in time of occurrence (time of day/night, month, or year)
  - Mode of dumping (abandoned containers, "midnight dumping" from moving vehicles, direct dumping of materials, accidents/spills)
  - Responsible parties
- Post "No Dumping" signs in problem areas with a phone number for reporting dumping and disposal. Signs should also indicate fines and penalties for illegal dumping.

Storm drain flushing is most effective in small diameter pipes (36-inch diameter pipe or less, depending on water supply and sediment collection capacity). Other considerations associated with storm drain flushing may include the availability of a water source, finding a downstream area to collect sediments, liquid/sediment disposal, and disposal of flushed effluent to sanitary sewer may be prohibited in some areas.

Maintenance

- Identifying illicit discharges requires teams of at least two people (volunteers can be used), plus administrative personnel, depending on the complexity of the storm sewer system.
- Arrangements must be made for proper disposal of collected wastes.
- Requires technical staff to detect and investigate illegal dumping violations, and to coordinate public education.

5.2.8.5 Example Schematic



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5.2.9 Dry Detention / Extended Detention Dry Basins

Detention
Structural Storm Water Control

Description: A surface storage basin or facility designed to provide water quantity control through detention and/or extended detention of storm water runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for the reduction of maximum runoff values associated with development to their pre-development levels.

ADVANTAGES / BENEFITS:

- Typically less costly than storm water (wet) ponds for equivalent flood storage, as less excavation is required
- Used in conjunction with water quality structural control
- Recreational and other open space opportunities between storm runoff events

DISADVANTAGES / LIMITATIONS:

- Controls for storm water quantity primarily –extended detention may provide limited water quality treatment and streambank protection

POLLUTANT REMOVAL

65%	**Total Suspended Solids**
50/30%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
No data	**Metals** - Cadmium, Copper, Lead, and Zinc removal
70%	**Pathogens** Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

S	**Water Quality Protection**
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
L	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *No restrictions.*

Soils: *Hydrologic group 'A' and 'B' soils may require pond liner*

Other Considerations:

- *Recreational and open space uses for dry detention*

L=Low **M**=Moderate **H**=High

5.2.9.1 General Description

Dry detention and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of storm water runoff to reduce downstream water quantity impacts. These facilities temporarily detain storm water runoff, releasing the flow over a period of time. They are designed to completely drain following a storm event and are normally dry between rain events.

Dry detention basins are intended to provide on-site flood control (peak flow reduction) and can be designed to control the extreme flood (100-year) storm event. Extended detention dry basins provide downstream streambank protection through extended detention of the streambank protection volume (SP_v), flood control.

Both dry detention and extended detention dry basins provide limited pollutant removal benefits and are not intended for water quality treatment. Detention-only facilities must be used in a treatment train approach with other structural controls that provide full treatment of the WQ_v (see Section 5.1).

Compatible multi-objective use of dry detention facilities in strongly encouraged.

5.2.9.2 Design Criteria and Specifications

Location

- Dry detention and extended detention dry basins are to be located downstream of other structural storm water controls providing treatment of the water quality volume (WQ_v). Extended detention dry basins may be part of a treatment train which treats the WQ_v. See Section 5.1 for more information on the use of multiple structural controls in a treatment train.

General Design

- Dry detention basins are sized to temporarily store the volume of runoff required to provide flood protection above the Q_f storm event up to the 100-year storm, if required.

  Extended detention dry basins are sized to provide extended detention of the streambank protection volume over 24 hours and can also provide additional storage volume for normal detention (peak flow reduction) of the 100-year storm event.

  Routing calculations must be used to demonstrate that the storage volume and outlet structure configuration are adequate. See Section 4.5 (*Storage Design*) for procedures on the design of detention storage.
- Storage may be subject to the requirements of the Texas Dam Safety Program (see Appendix H) based on the volume, dam height, and level of hazard.
- Earthen embankments less than 6 feet in height that are exposed to flood waters shall have side slopes no greater than the natural angle of repose of the fill material as determined by a geotechnical study. In lieu of a geotechnical study side slopes shall be 4:1 (horizontal to vertical) maximum.
- Earthen embankments 6 feet in height or greater shall be designed per Texas Commission on Environmental Quality guidelines for dam safety (see Appendix H).
- Vegetated slopes shall be less than 20 feet in height and shall have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred. Riprap-protected slopes shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for slopes greater than 10 feet in height.
- Areas above the normal high water elevations of the detention facility should be sloped toward the basin to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A low flow or pilot channel

across the facility bottom from the inlet to the outlet (often constructed with riprap) is recommended to convey low flows and prevent standing water conditions.

- Adequate maintenance access must be provided for all dry detention and extended detention dry basins.

Inlet and Outlet Structures

- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. A sediment forebay sized to 0.1 inches per impervious acre of contributing drainage should be provided for dry detention and extended detention dry basins that are in a treatment train with off-line water quality treatment structural controls.
- For a dry detention basin, the outlet structure is sized to its SP_v and Q_f functions (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. Small outlets that will be subject to clogging or are difficult to maintain are not acceptable.

  For an extended detention dry basin, a low flow orifice capable of releasing WQ_v and SP_v over 24 hours must be provided. The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (e.g., an overperforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

  See Section 4.6 (*Outlet Structures*) for more information on the design of outlet works.
- Seepage control or anti-seep collars should be provided for all outlet pipes.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Section 4.7, *Energy Dissipation Design*, for more guidance.
- An emergency spillway is to be included in the storm water pond design to safely pass the extreme flood flow. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be designed to State of Texas guidelines for dam safety (see Appendix H) and must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood, to the lowest point of the dam embankment not counting the emergency spillway.

5.2.9.3 Inspection and Maintenance Requirements

Table 5.2.9-1 Typical Maintenance Activities for Dry Detention / Extended Detention Dry Basins
(Source: Denver Urban Storm Drainage Manual, 1999)

Activity	Schedule
• Remove debris from basin surface to minimize outlet clogging and improve aesthetics.	Annually and following significant storm events
• Remove sediment buildup. • Repair and revegetate eroded areas. • Perform structural repairs to inlet and outlets.	As needed based on inspection
• Mow to limit unwanted vegetation.	Routine

5.2.9.4 Example Schematics

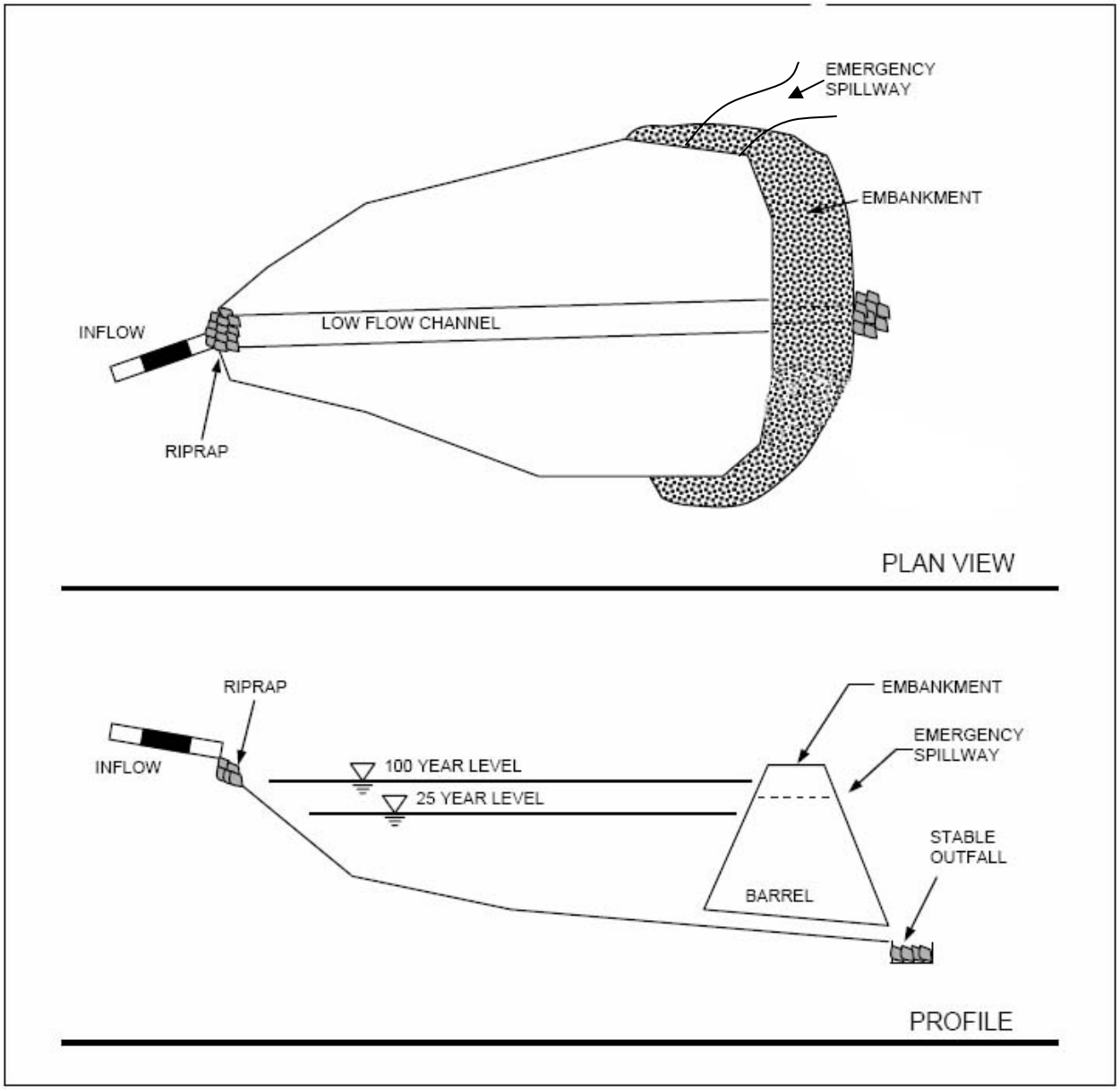

Figure 5.2.9-1 Schematic of Dry Detention Basin

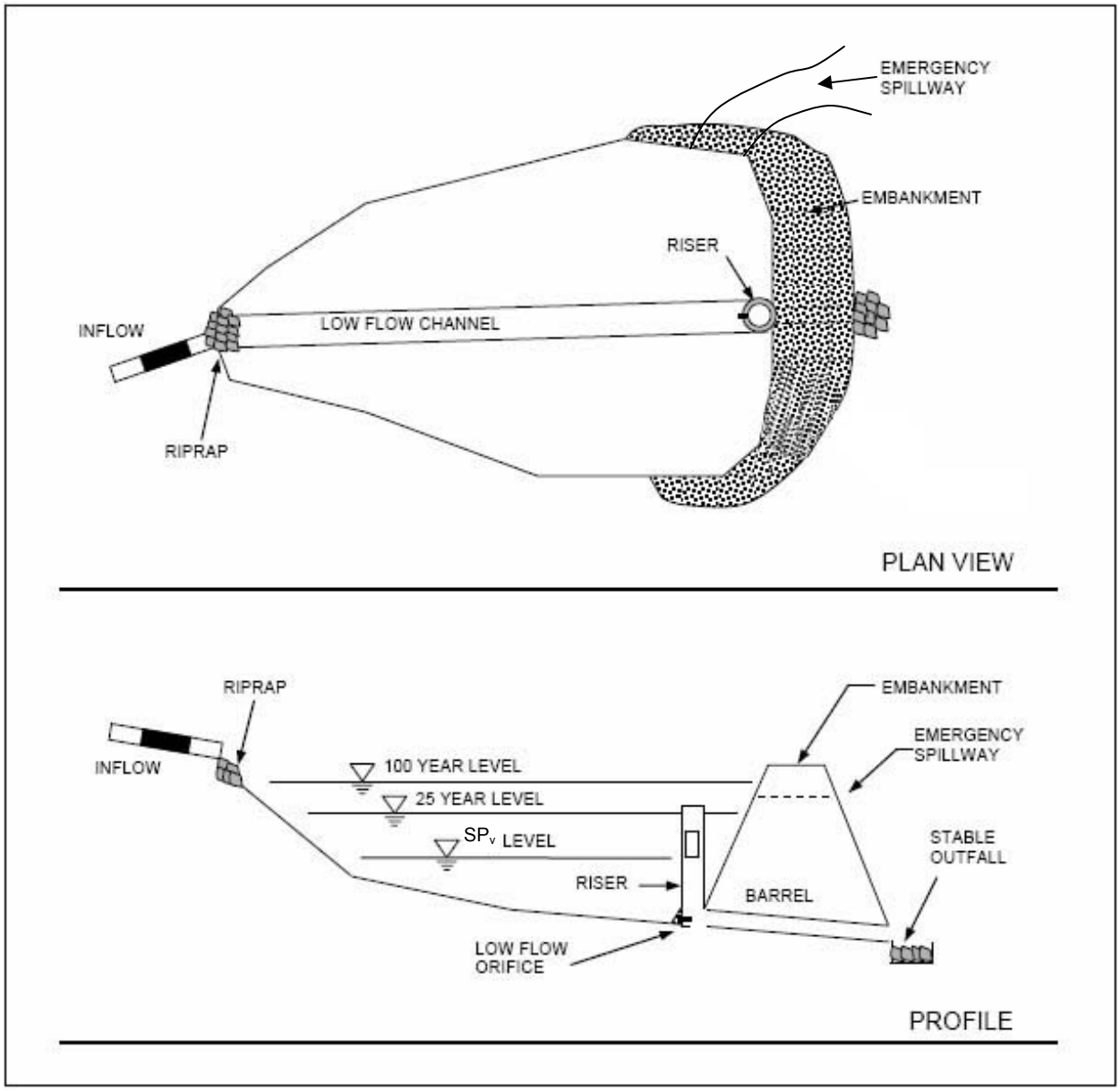

Figure 5.2.9-2 Schematic of Dry Extended Detention Basin

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5.2.10 Multi-Purpose Detention Areas

Structural Storm Water Control

Description: A facility designed primarily for another purpose, such as parking lots and rooftops that can provide water quantity control through detention of storm water runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Adequate grading and drainage must be provided to allow full use of facility's primary purposes following a storm event

ADVANTAGES / BENEFITS:

- Allows for multiple uses of site areas and reduces the need for downstream detention facilities
- Can be used in conjunction with water quality structural control

DISADVANTAGES / LIMITATIONS:

- Controls for storm water quantity only – not intended to provide water quality protection
- Localized flooding of area as intended may lead to property damage and additional liability

STORM WATER MANAGEMENT SUITABILITY

	Water Quality Protection
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**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: *No restrictions.*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.10.1 General Description

Multi-purpose detention areas are site areas primarily used for one or more specific activities that are also designed to provide for the temporary storage of storm water runoff to reduce downstream water quantity impacts. Example of multi-purpose detention areas include:

- Parking Lots

- Rooftops
- Sports Fields
- Recessed Plazas

Multi-purpose detention areas are normally dry between rain events, and by their very nature must be useable for their primary function the majority of the time. As such, multi-purpose detention areas should not be used for extended detention (SP_v control).

Multi-purpose detention areas are not intended for water quality protection and must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see Section 5.1).

5.2.10.2 Design Criteria and Specifications

Location

- Multi-purpose detention areas can be located upstream or downstream of other structural storm water controls providing treatment of the water quality protection volume (WQ_v). See Section 5.1 for more information on the use of multiple structural controls in a treatment train.

General Design

- Multi-purpose detention areas are sized to temporarily store a portion or all of the volume of runoff required to control the 100-year storm, if required.

  Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 4.5 (*Storage Design*) for procedures on the design of detention storage.
- All multi-purpose detention facilities must be designed to minimize potential safety risks, potential property damage, and inconvenience to the facility's primary purposes. Emergency overflows are to be provided for storm events larger than the design storm. The overflow must not create a significant adverse impact to downstream properties or the conveyance system.

Parking Lot Storage

- Parking lot detention can be implemented in areas where portions of large, paved lots can be temporarily used for runoff storage without significantly interfering with normal vehicle and pedestrian traffic. Parking lot detention can be created in two ways: by using ponding areas along sections of raised curbing, or through depressed areas of pavement at drop inlet locations.
- The maximum depth of detention ponding in a parking lot, except at a flow control structure, should be 6 inches for a 10-year storm, and 9 inches for a 100-year storm. The maximum depth of ponding at a flow control structure is 12 inches for a 100-year storm.
- The storage area (portion of the parking lot subject to ponding) must have a minimum slope of 0.5% towards the outlet to ensure complete drainage following a storm. A slope of 1% or greater is recommended.
- Fire lanes used for emergency equipment must be free of ponding water for runoff events up to the extreme storm (100-year) event.
- Flows are typically backed up in the parking lot using a wye inlet.

Rooftop Storage

- Rooftops can be used for detention storage as long as the roof support structure is designed to address the weight of ponded water and is sufficiently waterproofed to achieve a minimum service life

of 30 years. All rooftop detention designs must meet Texas State Building Code and local building code requirements.

- The minimum pitch of the roof area subject to ponding is 0.25 inches per foot.
- The rooftop storage system must include another mechanism for draining the ponding area in the event that the primary outlet is clogged.

Sports Fields

- Athletic facilities such as football and soccer fields and tracks can be used to provide storm water detention. This is accomplished by constructing berms around the facilities, which in essence creates very large detention basins. Outflow can be controlled through the use of an overflow weir or other appropriate control structure. Proper grading must be performed to ensure complete drainage of the facility.

Public Plazas

- In high-density areas, recessed public common areas such as plazas and pavilions can be utilized for storm water detention. These areas can be designed to flood no more than once or twice annually, and provide important open recreation space during the rest of the year.

5.2.10.3 Inspection and Maintenance Requirements

Table 5.2.10-1. Typical Maintenance Activities for Multi-Purpose Detention Areas

Activity	Schedule
• Remove debris from ponding area to minimize outlet clogging and improve aesthetics.	Annually and following significant storm events
• Remove sediment buildup. • Repair and revegetate eroded areas. • Perform structural repairs to inlet and outlets.	As needed based on inspection
• Perform additional maintenance activities specific to the type of facility.	As required

(Based on: Denver Urban Storm Drainage Manual, 1999)

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5.2.11 Underground Detention

Detention
Structural Storm Water Control

Description: Detention storage located in underground pipe/tank systems or vaults designed to provide water quantity control through detention and/or extended detention of storm water runoff.

KEY CONSIDERATIONS

ADVANTAGES / BENEFITS:

- Does not take up surface space
- Used in conjunction with water quality structural control
- Concrete vaults or pipe/tank systems can be used

DISADVANTAGES / LIMITATIONS:

- Controls for storm water quantity only – not intended to provide water quality treatment
- Intended for space-limited applications
- High initial construction cost as well as replacement cost at the end of its economic life

STORM WATER MANAGEMENT SUITABILITY

	Water Quality Protection
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
M	**Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *160 acres max.*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.11.1 General Description

Detention vaults are box-shaped underground storm water storage facilities typically constructed with reinforced concrete. Detention pipe/tank systems are underground storage facilities typically constructed with large diameter metal or plastic pipe. Both serve as an alternative to surface dry detention for storm water quantity control, particularly for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area.

Both underground vaults and pipe/tank systems can provide streambank protection through extended detention of the streambank protection volume (SP_v), and flood (in some cases extreme flood Q_f) control through normal detention. Basic storage design and routing methods are the same as for detention basins except that the bypass for high flows is typically included.

Underground detention vaults and pipe/tank systems are not intended for water quality treatment and must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see Section 5.1). This will prevent the underground vault or tank from becoming clogged with trash or sediment and significantly reduces the maintenance requirements for an underground detention system.

Prefabricated concrete vaults are available from commercial vendors. In addition, several pipe manufacturers have developed packaged detention systems.

5.2.11.2 Design Criteria and Specifications

Location

- Underground detention systems are to be located downstream of other structural storm water controls providing treatment of the water quality volume (WQ_v). See Section 5.1 for more information on the use of multiple structural controls in a treatment train.
- The maximum contributing drainage area to be served by a single underground detention vault or tank is 200 acres.

General Design

- Underground detention systems are sized to provide extended detention of the streambank protection volume over 24 hours and temporarily store the volume of runoff required to provide the desired flood protection.
- Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 4.5 (*Storage Design*) for procedures on the design of detention storage.
- Detention Vaults: Minimum 3,000 psi structural reinforced concrete may be used for underground detention vaults. All construction joints must be provided with water stops. Cast-in-place wall sections must be designed as retaining walls. The maximum depth from finished grade to the vault invert should be 20 feet.
- Detention Pipe/Tank Systems: The minimum pipe diameter for underground detention tanks is 36 inches.
- Underground detention vaults and pipe/tank systems must meet structural requirements for overburden support and traffic loading if appropriate.
- Adequate maintenance access must be provided for all underground detention systems. Access must be provided over the inlet pipe and outflow structure. Access openings can consist of a standard frame, grate and solid cover, or a removable panel. Vaults with widths of 10 feet or less should have removable lids.

Inlet and Outlet Structures

- A separate sediment sump or vault chamber sized to 0.1 inches per impervious acre of contributing drainage should be provided at the inlet for underground detention systems that are in a treatment train with off-line water quality treatment structural controls.
- For SP_v control, a low flow orifice capable of releasing the streambank protection volume over 24 hours must be provided. The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an overperforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

  For on-site flood control, an additional outlet is sized for control of the chosen return period (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure.

  See Section 4.6 (*Outlet Structures*) for more information on the design of outlet works.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. See Section 4.7, *Energy Dissipation Design*, for more guidance.
- A high flow bypass is to be included in the underground detention system design to safely pass the extreme flood flow.

5.2.11.3 Inspection and Maintenance Requirements

Table 5.2.11-1 Typical Maintenance Activities for Underground Detention Systems

Activity	Schedule
• Remove any trash/debris and sediment buildup in the underground vaults or pipe/tank systems.	Annually
• Perform structural repairs to inlet and outlets.	As needed, based on inspection

5.2.11.4 Example Schematics

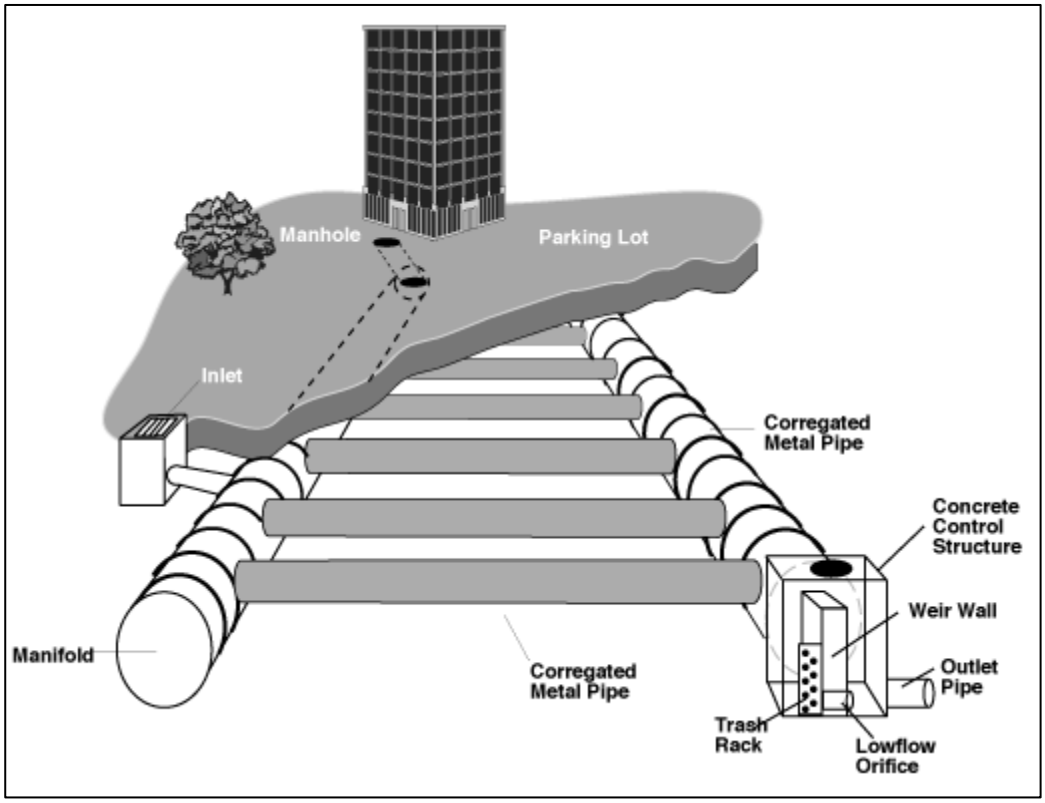

Figure 5.2.11-1 Example Underground Detention Tank System

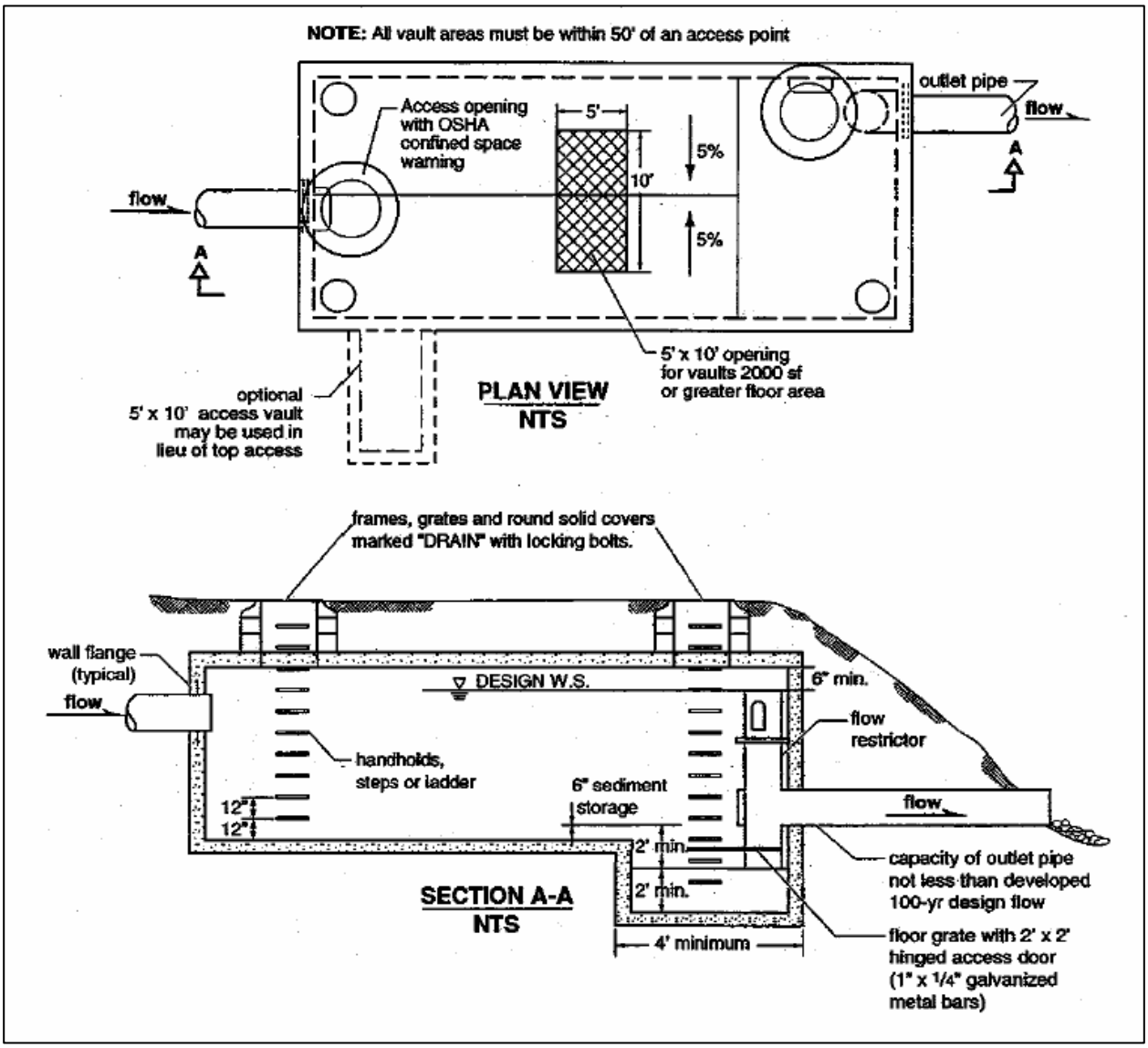

Figure 5.2.11-2 Schematic of Typical Underground Detention Vault
(Source: WDE, 2000)

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5.2.12 Filter Strip

Structural Storm Water Control

Description: Filter strips are uniformly graded and densely vegetated sections of land engineered and designed to treat runoff from and remove pollutants through vegetative filtering and infiltration.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Runoff from an adjacent impervious area must be evenly distributed across the filter strip as sheet flow

ADVANTAGES / BENEFITS:

- Can be used as part of the runoff conveyance system to provide pretreatment
- Can provide groundwater recharge
- Reasonably low construction cost

DISADVANTAGES / LIMITATIONS:

- Cannot alone achieve the 80% TSS removal target
- Large land requirement

MAINTENANCE REQUIREMENTS:

- Requires periodic repair, regrading, and sediment removal to prevent channelization

POLLUTANT REMOVAL

50%	**Total Suspended Solids**
20/20%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
40%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [S] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [H] **Land Requirement**
- [L] **Capital Cost**
- [M] **Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *2 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Use in buffer system*
- *Treating runoff from pervious areas*

L=Low **M**=Moderate **H**=High

5.2.12.1 General Description

Filter strips are uniformly graded and densely vegetated sections of land engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer, or as pretreatment for another structural storm water control. Filter strips can serve as a buffer between incompatible land uses, be landscaped to be aesthetically pleasing, and provide groundwater recharge in areas with pervious soils. Filter strips are often used as an *integrated* site design reduction credit (see Section 1.4 for more information).

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban storm water. There can also be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils while contained within the filter strip. To be effective, however, sheet flow must be maintained across the entire filter strip. Once runoff flow concentrates, it effectively short-circuits the filter strip and reduces any water quality benefits. Therefore, a flow spreader must normally be included in the filter strip design.

There are two different filter strip designs: a simple filter strip and a design that includes a permeable berm at the bottom. The presence of the berm increases the contact time with the runoff, thus reducing the overall width of the filter strip required to treat storm water runoff. Filter strips are typically an on-line practice, so they must be designed to withstand the full range of storm events without eroding.

5.2.12.2 Pollutant Removal Capabilities

Pollutant removal from filter strips is highly variable and depends primarily on density of vegetation and contact time for filtration and infiltration. These, in turn, depend on soil and vegetation type, slope, and presence of sheet flow.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- **Total Suspended Solids – 50%**
- **Total Phosphorus – 20%**
- **Total Nitrogen – 20%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 40%**

5.2.12.3 Design Criteria and Specifications

General Criteria

- Filter strips should be used to treat small drainage areas. Flow must enter the filter strip as sheet flow spread out over the width (long dimension normal to flow) of the strip, generally no deeper than 1 to 2 inches. As a rule, flow concentrates within a maximum of 75 feet for impervious surfaces, and 150 feet for pervious surfaces (CWP, 1996). For longer flow paths, special provision must be made to ensure design flows spread evenly across the filter strip, instead of becoming concentrated.
- Filter strips should be integrated within site designs.
- Filter strips should be constructed outside the natural stream buffer area whenever possible to maintain a more natural buffer along the streambank.
- Filter strips should be designed for slopes between 2% and 6%. Greater slopes than this would encourage the formation of concentrated flow. Flatter slopes would encourage standing water.

- Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance. Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and both wet and dry periods. See Appendix F for a list of appropriate grasses for use in North Central Texas.
- The flow path should be at least 15 feet across the strip to provide filtration and contact time for water quality treatment. Twenty-five (25) feet is preferred (where available), though the length of the flow path will normally be dictated by design method.
- Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.
- An effective flow spreader is a pea gravel diaphragm at the top of the slope (ASTM D 448 size no. 6, 1/8" to 3/8"). The pea gravel diaphragm (a small trench running along the top of the filter strip) serves two purposes. First, it acts as a pretreatment device, settling out sediment particles before they reach the practice. Second it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip. Other types of flow spreaders include a concrete sill, curb stops, or curb and gutter with "sawteeth" cut into it.
- Ensure that flows in excess of design flow move across or around the strip without damaging it. Often a bypass channel or overflow spillway with protected channel section is designed to handle higher flows.
- Pedestrian traffic across the filter strip should be limited through channeling onto sidewalks.
- Maximum discharge loading per foot of filter strip width (perpendicular to flow path) is found using the Manning's equation:

$$q = \frac{0.0237}{n} Y^{\frac{5}{3}} S^{\frac{1}{2}} \qquad \textbf{(5.2.12.1)}$$

where:

q = discharge per foot of width of filter strip (cfs/ft)
Y = allowable depth of flow (inches)
S = slope of filter strip (percent)
N = Manning's "n" roughness coefficient

(use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

- The minimum width of a filter strip is:

$$W_{fMIN} = \frac{Q}{q} \qquad \textbf{(5.2.12.2)}$$

where:

W_{fMIN} = minimum filter strip width perpendicular to flow (feet)

<u>Filter without Berm</u>

- Size filter strip (parallel to flow path) for a contact time of 5 minutes minimum
- Equation for filter length is based on the SCS TR55 travel time equation (SCS, 1986):

$$L_f = \frac{(T_t)^{1.25} (P_{2\text{-}24})^{0.625} (S)^{1/2}}{3.34n} \qquad \textbf{(5.2.12.3)}$$

where:

L_f = length of filter strip parallel to flow path (ft)

T_t = travel time through filter strip (minutes)
P_{2-24} = 2-year, 24-hour rainfall depth (inches)
S = slope of filter strip (percent)
n = Manning's "n" roughness coefficient

(use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

Filter Strips with Berm

- Size outlet pipes to ensure that the bermed area drains within 24 hours.
- Specify grasses resistant to frequent inundation within the shallow ponding limit.
- Berm material should be of sand, gravel and sandy loam to encourage grass cover (Sand: ASTM C-33 fine aggregate concrete sand 0.02"-0.04", Gravel: AASHTO M-43 ½" to 1").
- Size filter strip to contain the WQ_v within the wedge of water backed up behind the berm.
- Maximum berm height is 12 inches.

Filter Strips for Pretreatment

- A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a filter strip as a pretreatment measure. The required length of the filter strip flow path depends on the drainage area, imperviousness, and the filter strip slope. Table 5.2.12-1 provides sizing guidance for bioretention filter strips for pretreatment.

Table 5.2.12-1 Bioretention Filter Strip Sizing Guidance

Parameter	Impervious Areas				Pervious Areas (Lawns, etc)			
Maximum inflow approach length (feet)	35		75		75		100	
Filter strip slope (max = 6%)	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%
Filter strip minimum length (feet)	10	15	20	25	10	12	15	18

(Source: Claytor and Schueler, 1996)

5.2.12.4 Inspection and Maintenance Requirements

Table 5.2.12-2. Typical Maintenance Activities for Filter Strips

Activity	Schedule
• Mow grass to maintain a 2 to 4 inch height.	Regularly (frequently)
• Inspect pea gravel diaphragm for clogging and remove built-up sediment. • Inspect vegetation for rills and gullies and correct. Seed or sod bare areas. • Inspect to ensure that grass has established. If not, replace with an alternative species.	Annual Inspection (Semi-annual first year)

(Source: CWP, 1996)
If berm is used per example, inspect outlet pipes for clogging

Additional Maintenance Considerations and Requirements

- Filter strips require similar maintenance to other vegetative practices. Maintenance is very important for filter strips, particularly in terms of ensuring that flow does not short circuit the practice.

5.2.12.5 Example Schematic

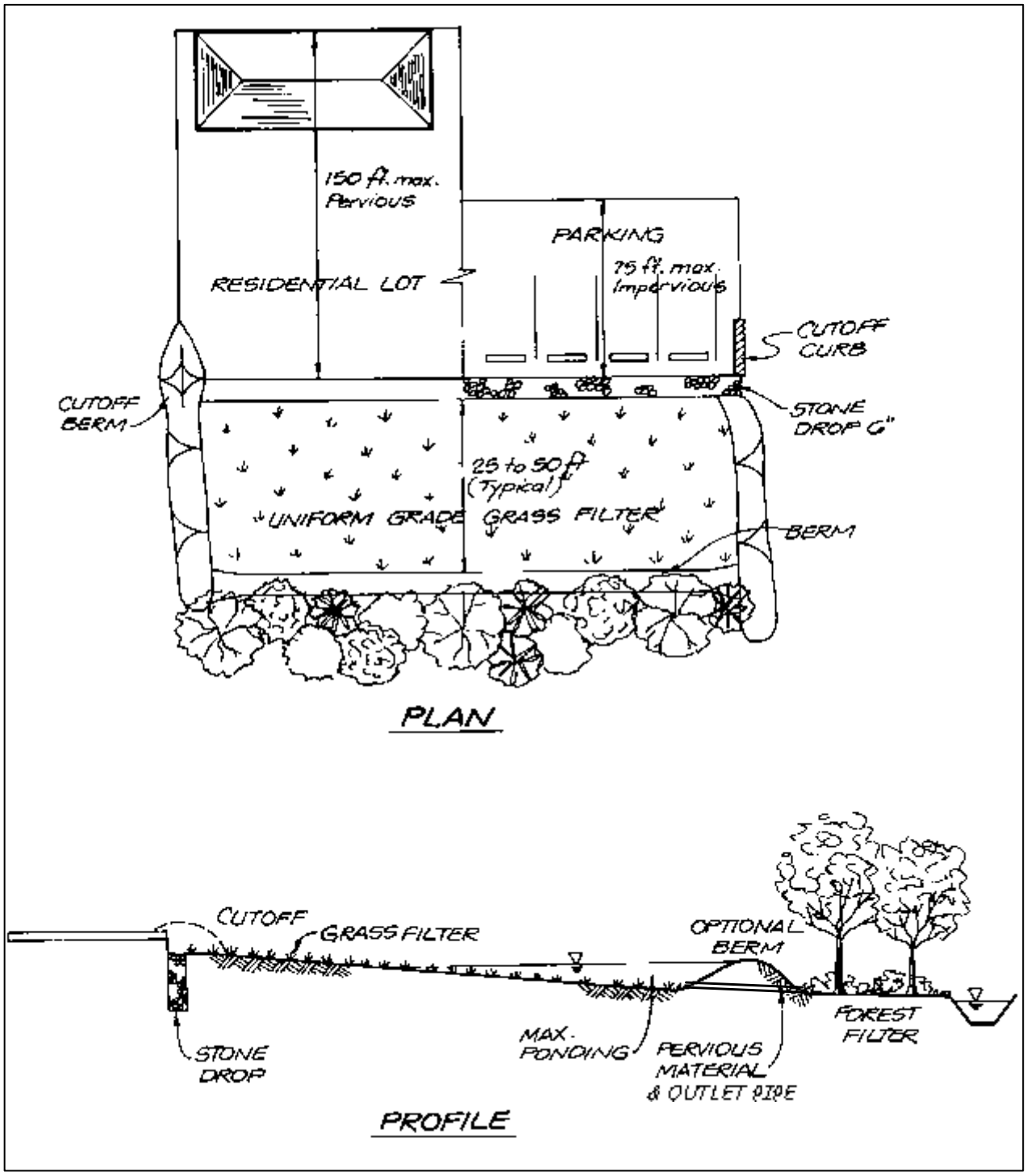

Figure 5.2.12-1 Schematic of Filter Strip (with Berm)

5.2.12.6 Design Example

Basic Data

Small commercial lot 150 feet deep x 100 feet wide located in Denison

- Drainage area (A) = 0.34 acres
- Impervious percentage (I) = 70%
- Slope equals 4%, Manning's n = 0.25

Calculate Maximum Discharge Loading Per Foot of Filter Strip Width

Using equation 5.2.12.1:

$q = 0.0237/0.25 * (1.0)^{5/3} * (4)^{1/2} = 0.19$ cfs/ft

Water Quality Peak Flow

See subsections 1.2.3 and 2.1.10.2 for details

Compute the Water Quality Volume in inches:

$WQ_v = 1.5 (0.05 + 0.009 * 70) = 1.02$ inches

Compute modified CN for 1.5-inch rainfall (P=1.5):

$$\begin{aligned} CN &= 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}] \\ &= 1000/[10+5*1.5+10*0.82-10(0.82^2+1.25*0.82*1.5)^{½}] \\ &= 92.4 \quad \text{(Use CN = 92)} \end{aligned}$$

For CN = 92 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.5-inch storm.

From Table 2.1.5-3, $I_a = 0.174$, therefore $I_a/P = 0.174/1.5 = 0.116$.

From Figure 2.1.5-6 for a Type II storm (using the limiting values) $q_u = 950$ csm/in, and therefore:

$Q_{wq} = (950 \text{ csm/in}) (0.34\text{ac}/640\text{ac/mi}^2) (1.02") = 0.51$ cfs

Minimum Filter Width

Using equation 5.3.2:

$W_{fMIN} = Q/q = 0.51/0.19 = 2.7$ feet

Since the width of the lot is 100 feet, the actual width of the filter strip will depend on site grading and the ability to deliver the drainage to the filter strip in sheet flow through a pea gravel filled trench.

Filter without Berm

- 2-year, 24-hour storm (see Appendix A) = 0.17 in/hr or 0.17*24= 4.08 inches
- Use 5 minute travel (contact) time

Using equation 5.2.12.3:

$L_f = (5)^{1.25} * (4.08)^{0.625} * (4)^{0.5} / (3.34 * 0.25) = 43$ feet

Note: Reducing the filter strip slope to 2% and planting a denser grass (raising the Manning n to 0.35) would reduce the filter strip length to 22 feet. Sensitivity to slope and Manning's n changes are illustrated for this example in Figure 5.2.12-2.

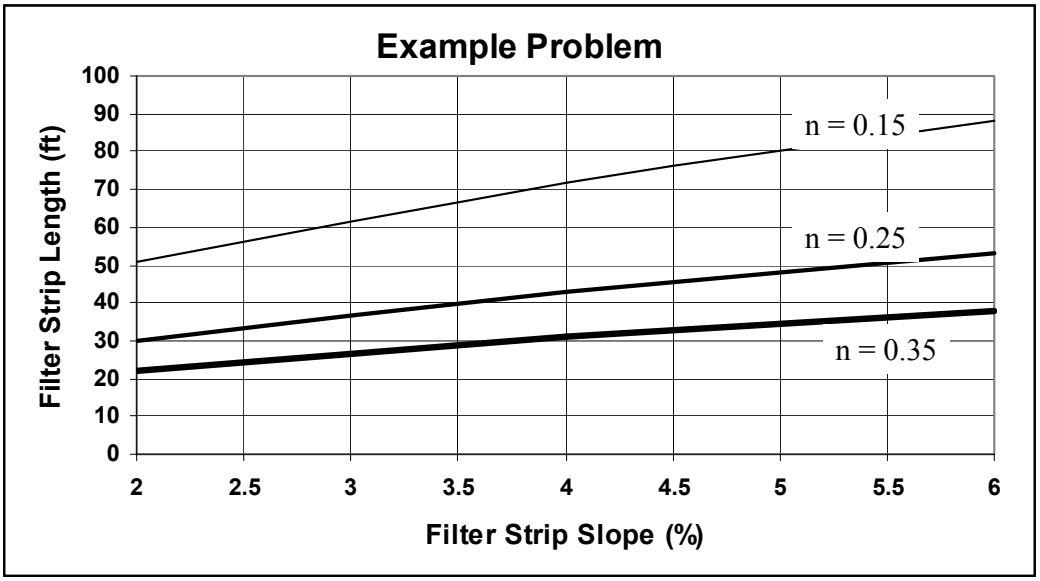

Figure 5.2.12-2 Example Problem Sensitivity of Filter Strip Length to Slope and Manning's n Values

Filter With Berm

- Pervious berm height is 6 inches

Compute the Water Quality Volume in cubic feet:

$WQ_v = Rv * 1.5/12 * A = (0.05 + 0.009 * 70) * 1.5/12 * 0.34 = 0.029$ Ac-ft or 1,259 ft^3

For a berm height of 6 inches the "wedge" of volume captured by the filter strip is:

Volume = $W_f * 0.5 * L_f * 0.5 = 0.25W_fL_f = 1{,}259\ ft^3$

For a maximum width of the filter of 100 feet, the length of the filter would then be 50 feet.

For a 1-foot berm height, the length of the filter would be 25 feet.

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5.2.13 Organic Filter

Structural Storm Water Control

Description: Design variant of the surface sand filter using organic materials in the filter media.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum head requirement of 5 to 8 feet

ADVANTAGES / BENEFITS:

- High pollutant removal capability
- Removal of dissolved pollutants is greater than sand filters due to cation exchange capacity

DISADVANTAGES / LIMITATIONS:

- Severe clogging potential if exposed soil surfaces exist upstream
- Intended for hotspot or space-limited applications, or for areas requiring enhanced pollutant removal capability
- High maintenance requirements
- Filter may require more frequent maintenance than most of the other storm water controls

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/40%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
75%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
50%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *10 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Hotspot areas*

L=Low **M**=Moderate **H**=High

5.2.13.1 General Description

The organic filter is a design variant of the surface sand filter, which uses organic materials such as leaf compost or a peat/sand mixture as the filter media. The organic material enhances pollutant removal by providing adsorption of contaminants such as soluble metals, hydrocarbons, and other organic chemicals.

As with the surface sand filter, an organic filter consists of a pretreatment chamber, and one or more filter cells. Each filter bed contains a layer of leaf compost or the peat/sand mixture, followed by filter fabric and a gravel/perforated pipe underdrain system. The filter bed and subsoils can be separated by an impermeable polyliner or concrete structure to prevent movement into groundwater.

Organic filters are typically used in high-density applications, or for areas requiring enhanced pollutant removal ability. Maintenance is typically higher than the surface sand filter facility due to the potential for clogging. In addition, organic filter systems have a higher head requirement than sand filters.

5.2.13.2 Pollutant Removal Capabilities

Peat/sand filter systems provide good removal of bacteria and organic waste metals. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. (Note: In some cases, organic materials may be a source of soluble phosphorus and nitrates.)

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 60%**
- **Total Nitrogen – 40%**
- **Fecal Coliform – 50%**
- **Heavy Metals – 75%**

5.2.13.3 Design Criteria and Specifications

- Organic filters are typically used on relatively small sites (up to 10 acres), to minimize potential clogging.
- The minimum head requirement (elevation difference needed at a site from the inflow to the outflow) for an organic filter is 5 to 8 feet.
- Organic filters can utilize a variety of organic materials as the filtering media. Two typical media bed configurations are the peat/sand filter and compost filter (see Figure 5.2.13-1). The peat filter includes an 18-inch 50/50 peat/sand mix over a 6-inch sand layer and can be optionally covered by 3 inches of topsoil and vegetation. The compost filter has an 18-inch compost layer. Both variants utilize a gravel underdrain system.
- The type of peat used in a peat/sand filter is critically important. Fibric peat in which undecomposed fibrous organic material is readily identifiable is the preferred type. Hemic peat containing more decomposed material may also be used. Sapric peat made up of largely decomposed matter should *not* be used in an organic filter.
- Typically, organic filters are designed as "off-line" systems, meaning that the water volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Storm water flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.
- Consult the design criteria for the surface sand filter (see subsection 5.2.15, *Sand Filters*) for the organic filter sizing and design steps.

5.2.13.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see subsection 5.2.15)

5.2.13.5 Example Schematic

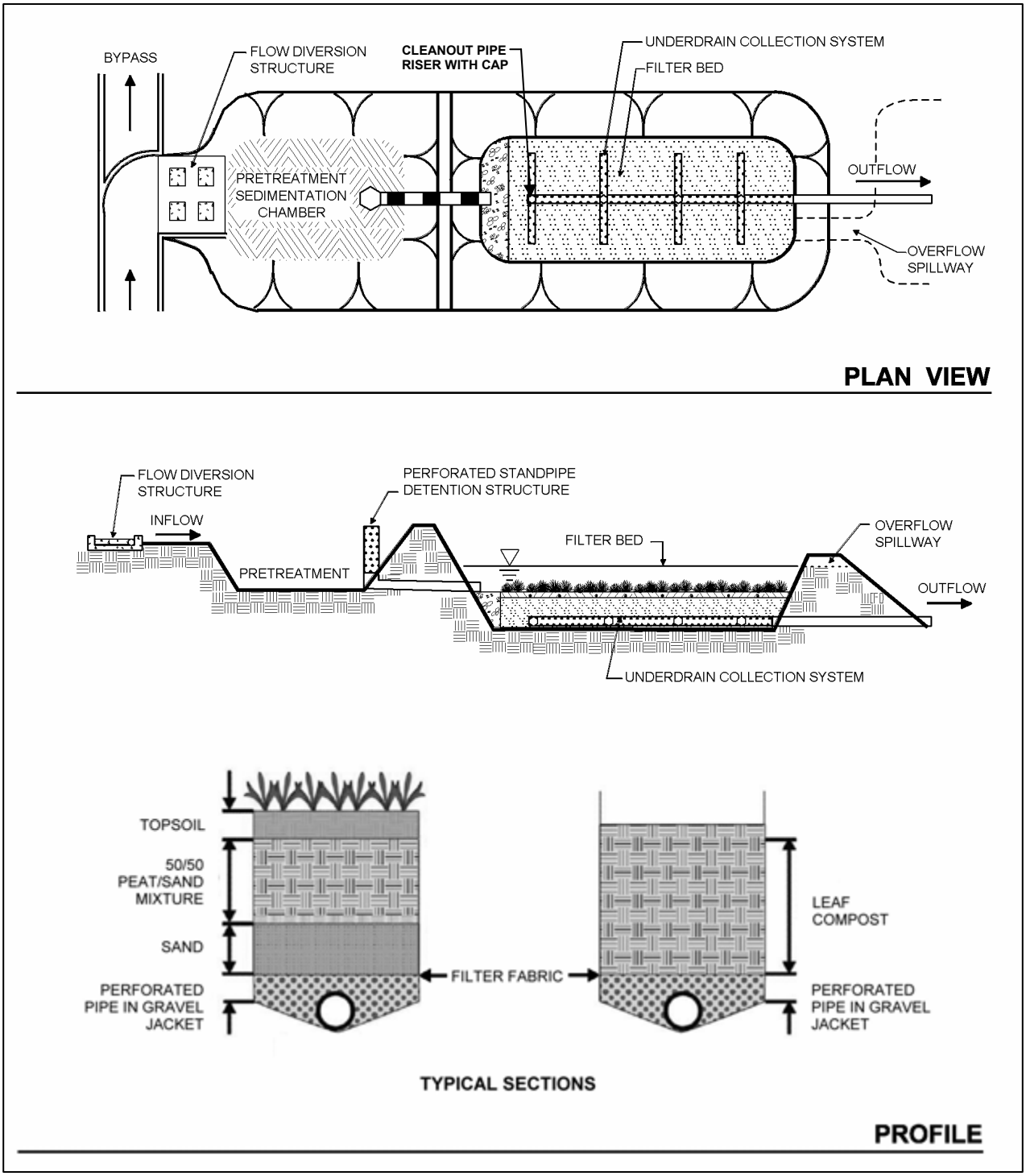

Figure 5.2.13-1 Schematic of Organic Filter
(Source: Center for Watershed Protection)

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5.2.14 Planter Boxes

Structural Storm Water Control

Description: Planter boxes are used on impervious surfaces in highly urbanized areas to collect and detain / infiltrate rainfall and runoff. The boxes may be prefabricated or constructed in place and contain growing medium, plants, and a reservoir.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Planter boxes should not be used for storm water containing high sediment loads to minimize clogging potential

ADVANTAGES / BENEFITS:

- Filtration provides pollutant removal capability
- Reservoir decreases peak flow rates

DISADVANTAGES / LIMITATIONS:

- Intended for space-limited applications, or for areas requiring additional pollutant removal capability
- Limited data on pollutant removal effectiveness

MAINTENANCE REQUIREMENTS:

- Vegetation will require frequent maintenance
- Filter may require more frequent maintenance than most of the other storm water controls

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/40%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
No data	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **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: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *No restrictions*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.14.1 General Description

Planter boxes are essentially large pots filled with soil or other growing media. There are several variations of this basic design. The contained planter box receives only rainfall, which filters through the soil and is then either taken up by its vegetation or allowed to seep out the bottom of the planter to the pavement or sidewalk. The infiltration planter box can receive both rainfall and runoff, which eventually filters through the bottomless planter and enters the underlying soil. The flow-through planter box collects flow in a perforated pipe along the bottom of the box and discharges out the side of the planter or into a storm sewer.

Each of the three planter box types has certain advantages and drawbacks:

- The contained planter is not tied into underlying soil or pipes and can therefore be placed almost anywhere and moved when needed. However, it does not have a reservoir to provide additional storage for flow control. Care should also be used in placing it next to building foundations and heavy pedestrian traffic areas.
- The infiltration planter should not be used next to foundations and underlying soils must drain rapidly enough to avoid ponding.
- The flow-through planter can be used next to building foundations since it directs flow off to the side and away from the building. It must be located next to a suitable discharge point into the storm water conveyance system.

5.2.14.2 Pollutant Removal Capabilities

Field tests of planters are lacking, however, tests of a bioretention cell by the EPA showed results that were generally similar to those of the Organic Filter, with somewhat less metals removal (43-78%).

5.2.14.3 Design Criteria and Specifications

- The infiltration and flow-through planter boxes can capture runoff from surrounding areas and provide limited storage in reservoirs. The ratio of planter area to impervious area should be 7%, assuming a storm volume of 1.5 inches and a reservoir depth in the planter of 12 inches.
- The planter should be constructed of stone, concrete, or brick. Pressure-treated wood may be used if it does not leach out toxic chemicals that might contaminate storm water.
- Filter media should consist of sand, gravel and topsoil as shown in the figures below. As an alternative, compost/mulch can be used in place of the sand, gravel, and topsoil, but will have different infiltration characteristics. Compost with organics will aid in pollutant removal through absorption, but it will remove nitrogen from the plant material as it breaks down/decomposes. A nitrogen fertilizer may need to be added should this occur.
- Planter vegetation should be relatively self-sustaining, with minimal fertilizer or pesticide requirements. Grasses, herbs, succulents, shrubs, and trees may be used in planter boxes. Examples include rushes, reeds, sedges, iris, dogwood, currants, and other approved species. Trees are encouraged as their foliage traps additional precipitation.
- All of the planters require 18 inches of growing media. The contained planter does not require a minimum width. A minimum width of 30 inches is recommended for the infiltration planter. The flow-through planter should be at least 18 inches wide. The minimum widths help reduce water wicking down the insides of the planter wall.
- Water should drain through a planter within 3-4 hours after the storm event.
- Soils underneath an infiltration planter should be SCS Hydrologic Type A or B.

5.2.14.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for planter boxes focus on maintaining an adequate drainage rate through the planting media and attractive and healthy vegetation.

Table 5.2.14-1 Typical Maintenance Activities for Planter Boxes

Activity	Schedule
• Ensure that downspout or sheet flow from paving is unimpeded. • Ensure planter reservoir drains within 3-4 hours. • Replace or amend topsoil if drainage unsatisfactory.	Quarterly and within 48 hours of major storms
• Ensure that contributing area and planter boxes are clear of debris. • Remove accumulated sediment if greater than 4 inches in depth. • Ensure that planter vegetation is healthy and planter is weeded and shrubs and trees pruned. • Planter vegetation may require watering during long dry spells.	As needed, based on inspection
• Fallen leaves and debris from deciduous plants should be removed.	Three to four times a year
• Replenish mulch. • Training/written materials provided to property owners and tenants.	Annually
• Replace planter if cracked or rotted.	Upon failure

5.2.14.5 Example Schematics

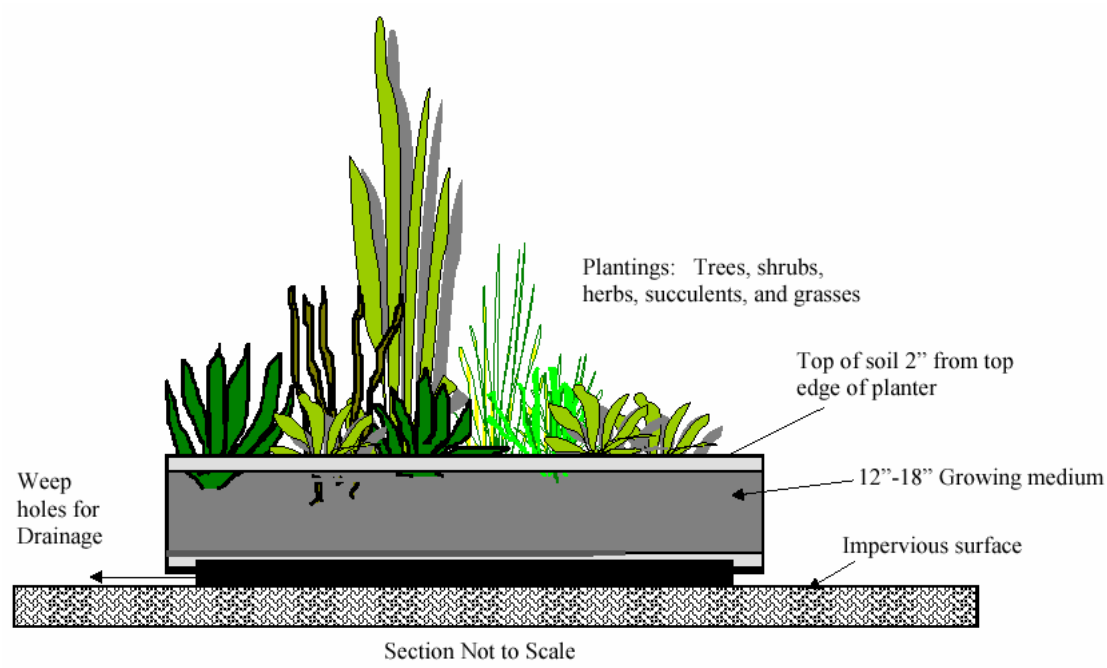

Figure 5.2.14-1 Schematic of Contained Planter Box
(Source: City of Portland, Oregon)

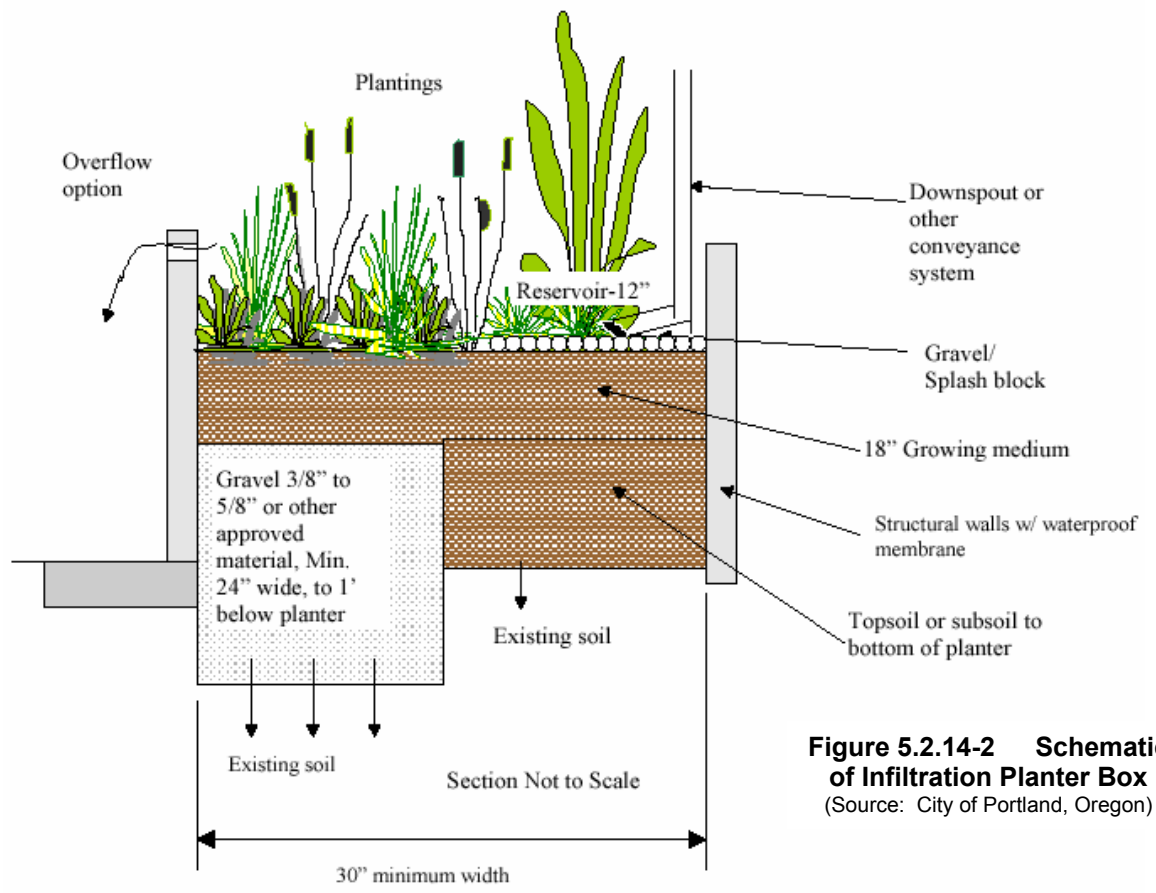

Figure 5.2.14-2 Schematic of Infiltration Planter Box
(Source: City of Portland, Oregon)

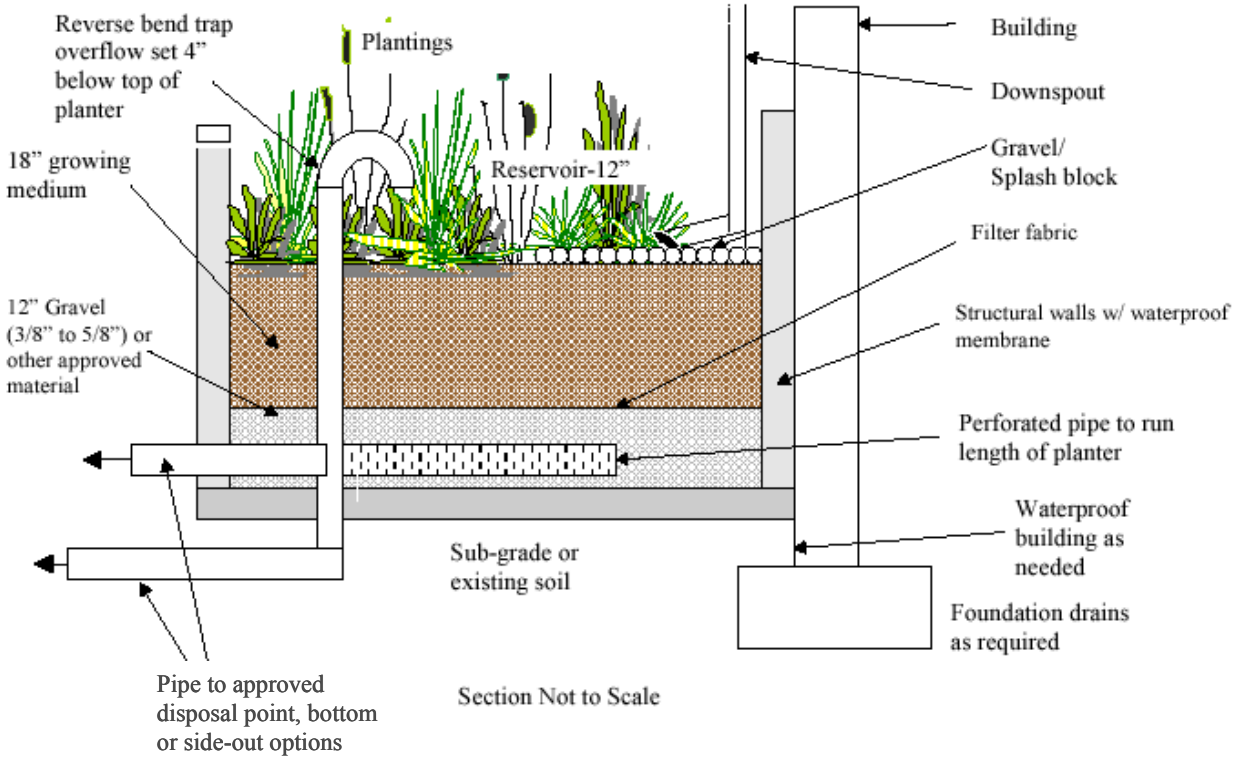

Figure 5.2.14-3 Schematic of Flow-Through Planter Box
(Source: City of Portland, Oregon)

5.2.15 Sand Filters

General Application
Structural Storm Water Control

Description: Multi-chamber structure designed to treat storm water runoff through filtration, using a sediment forebay, a sand bed as its primary filter media and, typically, an underdrain collection system.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Typically requires 2 to 6 feet of head
- Maximum contributing drainage area of 10 acres for surface sand filter; 2 acres for perimeter sand filter
- Sand filter media with underdrain system

ADVANTAGES / BENEFITS:

- Applicable to small drainage areas
- Good for highly impervious areas
- Good retrofit capability

DISADVANTAGES / LIMITATIONS:

- High maintenance burden
- Not recommended for areas with high sediment content in storm water or clay/silt runoff areas
- Relatively costly
- Possible odor problems

MAINTENANCE REQUIREMENTS:

- Inspect for clogging – rake first inch of sand
- Remove sediment from forebay/chamber
- Replace sand filter media as needed

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/25%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
40%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
S	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *2-10 acres max.*

Soils: *Clay or silty soils may require pretreatment*

Other Considerations:

- Typically needs to be combined with other controls to provide water quantity control

L=Low **M**=Moderate **H**=High

5.2.15.1 General Description

Sand filters (also referred to as *filtration basins*) are structural storm water controls that capture and temporarily store storm water runoff and pass it through a filter bed of sand. Most sand filter systems consist of two-chamber structures. The first chamber is a sediment forebay or sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes additional pollutants by filtering the runoff through a sand bed. The filtered runoff is typically collected and returned to the conveyance system, though it can also be partially or fully infiltrated into the surrounding soil in areas with porous soils.

Because they have few site constraints beside head requirements, sand filters can be used on development sites where the use of other structural controls may be precluded. However, sand filter systems can be relatively expensive to construct, install, and maintain.

There are two primary sand filter system designs, the *surface sand filter* and the *perimeter sand filter*. Below are descriptions of these filter systems:

- **Surface Sand Filter** – The surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. This system can treat drainage areas up to 10 acres in size and is typically located off-line. Surface sand filters can be designed as an excavation with earthen embankments or as a concrete or block structure.
- **Perimeter Sand Filter** – The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the control.

A third design variant, the *underground sand filter*, is intended primarily for extremely space limited and high density areas and is thus considered a limited application structural control. See subsection 5.2.16 for more details.

Surface Sand Filter

Perimeter Sand Filter

Figure 5.2.15-1 Sand Filter Examples

5.2.15.2 Storm Water Management Suitability

Sand filter systems are designed primarily as off-line systems for storm water quality (i.e., the removal of storm water pollutants) and will typically need to be used in conjunction with another structural control to provide downstream streambank protection, on-site flood control, and downstream flood control, if required. However, under certain circumstances, filters can provide limited runoff quantity control, particularly for smaller storm events.

Water Quality

In sand filter systems, storm water pollutants are removed through a combination of gravitational settling, filtration, and adsorption. The filtration process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants. Surface sand filters with a grass cover have additional opportunities for bacterial decomposition as well as vegetation uptake of pollutants, particularly nutrients. Section 5.2.15.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

For smaller sites, a sand filter may be designed to capture the entire streambank protection volume SP_v in either an off- or on-line configuration. Given that a sand filter system is typically designed to completely drain over 40 hours, the requirement of extended detention of the 1-year, 24-hour storm runoff volume will be met. For larger sites or where only the WQ_v is diverted to the sand filter facility, another structural control must be used to provide SP_v extended detention.

On-Site Flood Control

Another structural control must be used in conjunction with a sand filter system to reduce the post-development peak flow to pre-development levels (detention) if needed.

Downstream Flood Control

Sand filter facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

The volume of runoff removed and treated by the sand filter may be taken in the on-site flood control and downstream flood control calculations (see Section 5.1).

5.2.15.3 Pollutant Removal Capabilities

Both the surface and perimeter sand filters are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed sand filters can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 50%**
- **Total Nitrogen – 25%**
- **Fecal Coliform – 40%**
- **Heavy Metals – 50%**

For additional information and data on pollutant removal capabilities for sand filters, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Storm Water Best Management Practices (BMP) Database at www.bmpdatabase.org

5.2.15.4 Application and Site Feasibility Criteria

Sand filter systems are well suited for highly impervious areas where land available for structural controls is limited. Sand filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards. Sand filters may also be feasible and appropriate in some multi-family or higher density residential developments.

To avoid rapid clogging and failure of the filter media, the use of sand filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of a sand filter facility for meeting storm water management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – NO
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Storm Water Control – NO

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 10 acres maximum for surface sand filter; 2 acres maximum for perimeter sand filter
- Space Required – Function of available head at site
- Site Slope – No more than 6% slope across filter location
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 5 feet for surface sand filters; 2 to 3 feet for perimeter sand filters
- Minimum Depth to Water Table – For a surface sand filter with infiltration (earthen structure), 2 feet are required between the bottom of the sand filter and the elevation of the seasonally high water table
- Soils – No restrictions; Group "A" soils generally required to allow infiltration (for surface sand filter earthen structure)
- Downstream Water Surface - Downstream flood conditions need to be verified to avoid surcharging and back washing of the filter material.

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

5.2.15.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a sand filter facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- Surface sand filters should have a contributing drainage area of 10 acres or less. The maximum drainage area for a perimeter sand filter is 2 acres.

Sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sites with less than 50% imperviousness or high clay/silt sediment loads must not use a sand filter without adequate pretreatment due to potential clogging and failure of the filter bed. Any disturbed areas

within the sand filter facility drainage area should be identified and stabilized. Filtration controls should only be constructed after the construction site is stabilized.

- Surface sand filters are generally used in an off-line configuration where the water quality volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Storm water flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.
- Perimeter sand filters are typically sited along the edge, or perimeter, of an impervious area such as a parking lot.
- Sand filter systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

B. GENERAL DESIGN

Surface Sand Filter

- A surface sand filter facility consists of a two-chamber open-air structure, which is located at ground-level. The first chamber is the sediment forebay (a.k.a sedimentation chamber) while the second chamber houses the sand filter bed. Flow enters the sedimentation chamber where settling of larger sediment particles occurs. Runoff is then discharged from the sedimentation chamber through a perforated standpipe into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure 5.2.15-6 provides plan view and profile schematics of a surface sand filter.

Perimeter Sand Filter

- A perimeter sand filter facility is a vault structure located just below grade level. Runoff enters the device through inlet grates along the top of the structure into the sedimentation chamber. Runoff is discharged from the sedimentation chamber through a weir into the filtration chamber. After passing through the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure 5.2.15-7 provides plan view and profile schematics of a perimeter sand filter.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

Surface Sand Filter

- The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure 5.2.15-2 illustrates the distribution of the treatment volume (0.75 WQ_v) among the various components of the surface sand filter, including:
  - V_s – volume within the sedimentation basin
  - V_f – volume within the voids in the filter bed
  - $V_{f\text{-}temp}$ – temporary volume stored above the filter bed
  - A_s – the surface area of the sedimentation basin
  - A_f – surface area of the filter media
  - h_s – height of water in the sedimentation basin
  - h_{temp} - depth of temporary volume
  - h_f – average height of water above the filter media (1/2 h_{temp})
  - d_f – depth of filter media

- The sedimentation chamber must be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of at least 2:1. Inlet and outlet structures should be located at opposite ends of the chamber.

The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.

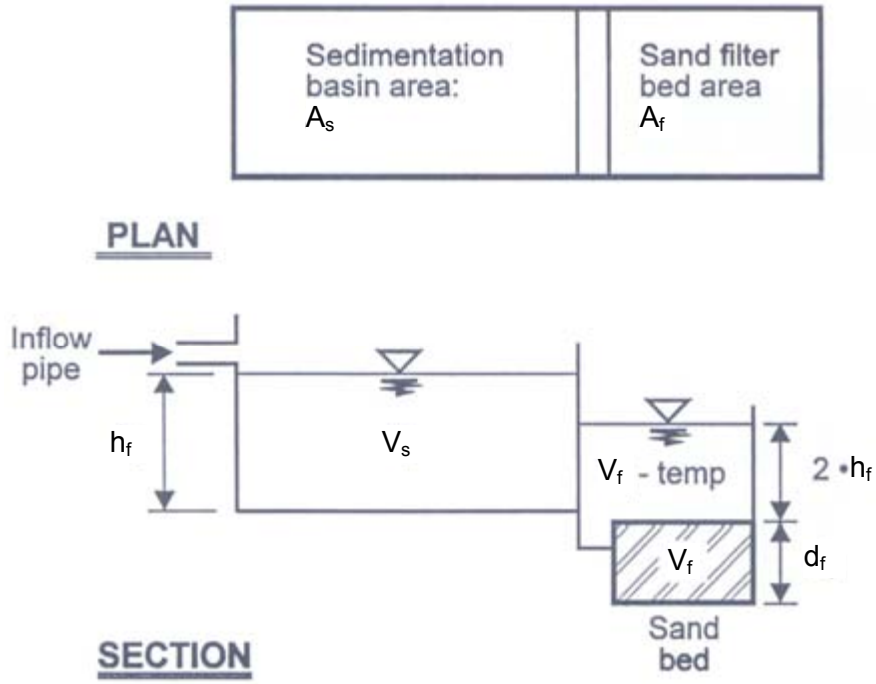

Figure 5.2.15-2 Surface Sand Filter Volumes
Source: Claytor and Schueler, 1996

- The filter media consists of an 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand or TxDOT Fine Aggregate Grade No. 1) on top of the underdrain system. Three inches of topsoil are placed over the sand bed. Permeable filter fabric is placed both above and below the sand bed to prevent clogging of the sand filter and the underdrain system. A proper fabric selection is critical. Choose a filter fabric with equivalent pore openings as to prevent clogging by sandy filler material. Figure 5.2.15-4 illustrates a typical media cross section.
- The filter bed is equipped with a 6-inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8-inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40% meeting the gradation listed below. Aggregate contaminated with soil shall not be used.

Gradation

Sieve Size	% Passing
2 ½"	100
2"	90 – 100
1 ½"	35 – 70
1"	0 – 15
½"	0 - 5

- The structure of the surface sand filter may be constructed of impermeable media such as concrete, or through the use of excavations and earthen embankments. When constructed with earthen walls/embankments, filter fabric should be used to line the bottom and side slopes of the structures before installation of the underdrain system and filter media.

Perimeter Sand Filter

- The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure 5.2.15-3 illustrates the distribution of the treatment volume (0.75 WQ_v) among the various components of the perimeter sand filter, including:
  - V_w – wet pool volume within the sedimentation basin
  - V_f – volume within the voids in the filter bed
  - V_{temp} – temporary volume stored above the filter bed
  - A_s – the surface area of the sedimentation basin
  - A_f – surface area of the filter media
  - h_f – average height of water above the filter media (1/2 h_{temp})
  - h_{temp} - depth of temporary volume
  - d_f – depth of filter media
- The sedimentation chamber must be sized to at least 50% of the computed WQ_v.
- The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.
- The filter media should consist of a 12- to 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand or TxDOT Fine Aggregate Grade No. 1) on top of the underdrain system. Figure 5.2.15-4 illustrates a typical media cross section.
- The perimeter sand filter is equipped with a 4 inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8 inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. A permeable filter fabric should be placed between the gravel layer and the filter media. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40% meeting the following gradation. Aggregate contaminated with soil shall not be used.

Gradation

Sieve Size	% Passing
2 ½"	100
2"	90 – 100
1 ½"	35 – 70
1"	0 – 15
½"	0 - 5

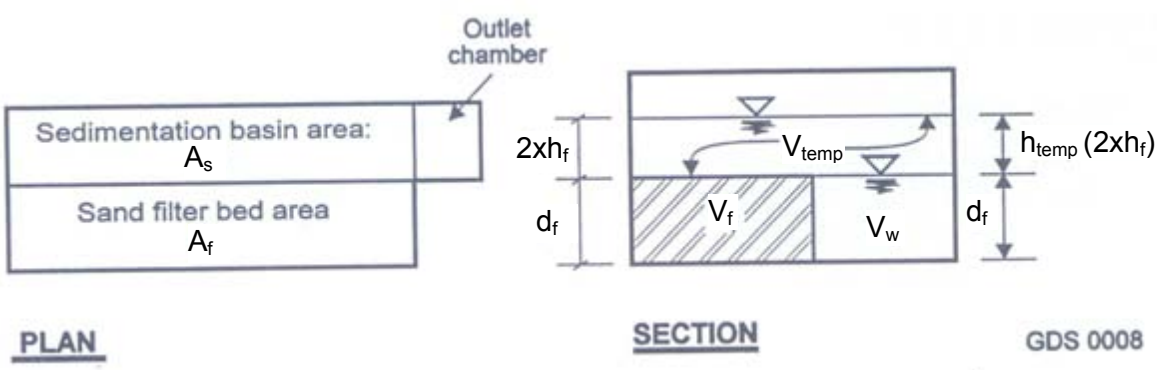

Figure 5.2.15-3 Perimeter Sand Filter Volumes
(Source: Claytor and Schueler, 1996)

D. PRETREATMENT / INLETS

- Pretreatment of runoff in a sand filter system is provided by the sedimentation chamber.
- Inlets to surface sand filters are to be provided with energy dissipaters. Exit velocities from the sedimentation chamber must be nonerosive.
- Figure 5.2.15-5 shows a typical inlet pipe from the sedimentation basin to the filter media basin for the surface sand filter.

E. OUTLET STRUCTURES

- Outlet pipe is to be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways).

F. EMERGENCY SPILLWAY

- An emergency or bypass spillway must be included in the surface sand filter to safely pass flows that exceed the design storm flows. The spillway prevents filter water levels from overtopping the embankment and causing structural damage. The emergency spillway should be located so that downstream buildings and structures will not be impacted by spillway discharges.

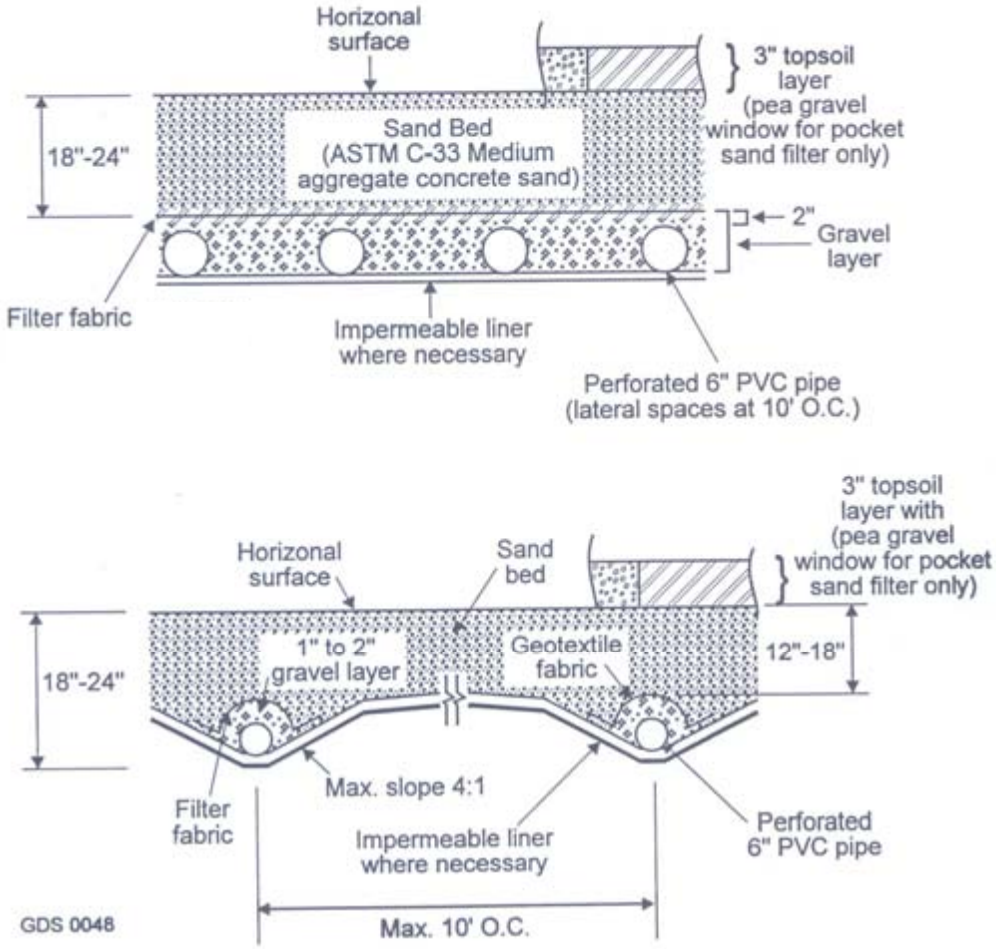

Figure 5.2.15-4 Typical Sand Filter Media Cross Sections
(Source: Claytor and Schueler, 1996)

G. MAINTENANCE ACCESS

- Adequate access must be provided for all sand filter systems for inspection and maintenance, including the appropriate equipment and vehicles. Access grates to the filter bed need to be included in a perimeter sand filter design. Facility designs must enable maintenance personnel to easily replace upper layers of the filter media.

H. SAFETY FEATURES

- Surface sand filter facilities can be fenced to prevent access. Inlet and access grates to perimeter sand filters may be locked.

I. LANDSCAPING

- Surface filters can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of inundation and drought.

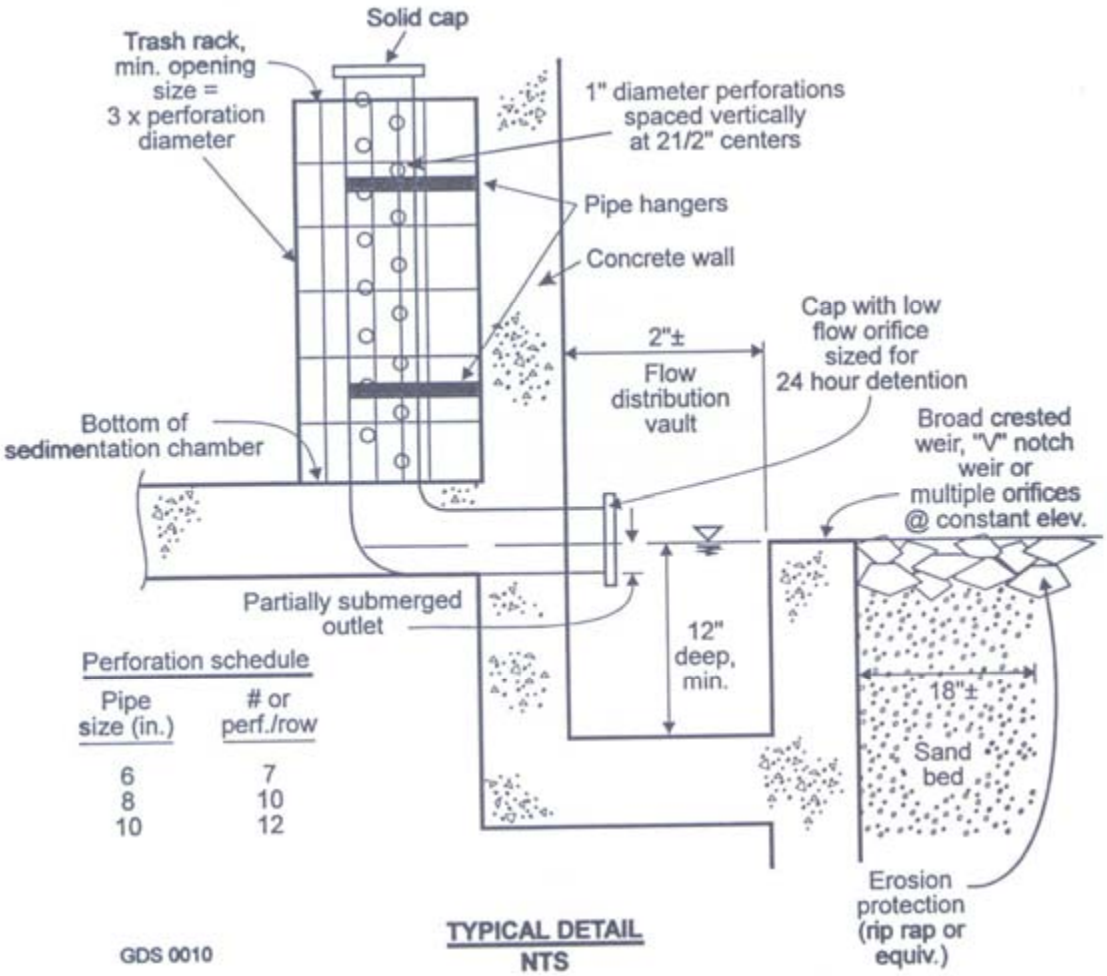

Figure 5.2.15-5 Surface Sand Filter Perforated Stand-Pipe
(Source: Claytor and Schueler, 1996)

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of surface sand filter may be limited by low head
- High Relief – Filter bed surface must be level
- Karst – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure

Soils

- No restrictions

Special Downstream Watershed Considerations

- Stream Warming – Consideration should be given to the thermal influence on potential fish habitats downstream. If stream warming is significant, use shorter drain time (24 hours)
- Aquifer Protection – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure; no infiltration of filter runoff into groundwater

5.2.15.6 Design Procedures

Step 1. Compute runoff control volumes from the *integrated* Design Approach

Calculate the Water Quality Volume (WQ_v), Streambank Protection Volume (SP_v), On-Site Flood Control Volume (Q_p), and the Downstream Flood Control Volume (V_f).

Details on the *integrated* Design Approach are found in Section 1.2.

Step 2. Determine if the development site and conditions are appropriate for the use of a surface or perimeter sand filter.

Consider the Application and Site Feasibility Criteria in subsections 5.2.15.4 and 5.2.15.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 5.2.15.5-J (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see subsection 2.1.10).

(a) Using WQ_v, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5. Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the sand filter facility.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6. Size filtration basin chamber

The filter area is sized using the following equation (based on Darcy's Law):

$$\mathbf{A_f = (WQ_v)\,(d_f) / [(k)\,(h_f + d_f)\,(t_f)]}$$

where:

A_f = surface area of filter bed (ft^2)
d_f = filter bed depth
(typically 18 inches, no more than 24 inches)
k = coefficient of permeability of filter media (ft/day)
(use 3.5 ft/day for sand)
h_f = average height of water above filter bed (ft)
(1/2 h_{max}, which varies based on site but h_{max} is typically $\leq$ 6 feet)
t_f = design filter bed drain time (days)
(1.67 days or 40 hours is recommended maximum)

Set preliminary dimensions of filtration basin chamber.

See subsection 5.2.15.5-C (Physical Specifications/Geometry) for filter media specifications.

Step 7. Size sedimentation chamber

Surface sand filter: The sedimentation chamber should be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$\mathbf{A_s = -(Q_o/w) * Ln\,(1-E)}$$

Where:

A_s = sedimentation basin surface area (ft^2)
Q_o = rate of outflow = the WQ_v over a 24-hour period
w = particle settling velocity (ft/sec)
E = trap efficiency

Assuming:

- 90% sediment trap efficiency (0.9)
- particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness < 75%
- particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness $\geq$ 75%
- average of 24 hour holding period

Then:

A_s = (0.066) (WQ_v) ft^2 for I < 75%
A_s = (0.0081) (WQ_v) ft^2 for I $\geq$ 75%

Set preliminary dimensions of sedimentation chamber.

Perimeter sand filter: The sedimentation chamber should be sized to at least 50% of the computed WQ_v. Use same approach as for surface sand filter.

Step 8. Compute V_{min}

$$V_{min} = 0.75 * WQ_v$$

Step 9. Compute storage volumes within entire facility and sedimentation chamber orifice size

Surface sand filter:

$$V_{min} = 0.75\ WQ_v = V_s + V_f + V_{f\text{-}temp}$$

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
Where: n = porosity = 0.4 for most applications

(2) Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = $2 * h_f * A_f$

(3) Compute V_s = volume within sediment chamber = $V_{min} - V_f - V_{f\text{-}temp}$

(4) Compute h_s = height in sedimentation chamber = V_s/A_s

(5) Ensure h_s and h_f fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.

(6) Size orifice from sediment chamber to filter chamber to release V_s within 24-hours at average release rate with 0.5 h_s as average head.

(7) Design outlet structure with perforations allowing for a safety factor of 10 (see example)

(8) Size distribution chamber to spread flow over filtration media – level spreader weir or orifices.

Perimeter sand filter:

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
Where: n = porosity = 0.4 for most applications

(2) Compute V_w = wet pool storage volume A_s * 2 feet minimum

(3) Compute V_{temp} = temporary storage volume = $V_{min} – (V_f + V_w)$

(4) Compute h_{temp} = temporary storage height = $V_{temp} / (A_f + A_s)$

(5) Ensure $h_{temp} \geq 2 * h_f$, otherwise decrease h_f and re-compute. Ensure dimensions fit available head and area – change as necessary in design iterations until all site dimensions fit.

(6) Size distribution slots from sediment chamber to filter chamber.

Step 10. Design inlets, pretreatment facilities, underdrain system, and outlet structures

See subsection 5.2.15-5-D through H for more details.

Step 11. Compute overflow weir sizes

Surface sand filter:

1. Size overflow weir at elevation h_s in sedimentation chamber (above perforated stand pipe) to handle surcharge of flow through filter system from storms producing more than 1.5 inches (see example in Appendix D).

2. Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation h_f in filtration chamber (above perforated stand pipe) to handle surcharge of flow through filter system from storms producing more than 1.5 inches (see example).

Perimeter sand filter: Size overflow weir at end of sedimentation chamber to handle excess inflow, set at WQ_v elevation.

See Appendix D-3 for a Sand Filter Design Example

5.2.15.7 Inspection and Maintenance Requirements

Table 5.2.15-1 Typical Maintenance Activities for Sand Filters
(Source: WMI, 1997; Pitt, 1997)

Activity	Schedule
• Ensure that contributing area, facility, inlets, and outlets are clear of debris. • Ensure that the contributing area is stabilized and mowed, with clippings removed. • Remove trash and debris. • Check to ensure that the filter surface is not clogging (also check after moderate and major storms). • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. • If permanent water level is present (perimeter sand filter), ensure that the chamber does not leak and normal pool level is retained.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Stabilize disturbed area contributing to the heavy sediment load. • Make sure that there is no evidence of deterioration, spalling, or cracking of concrete. • Inspect grates (perimeter sand filter). • Inspect inlets, outlets, and overflow spillway to ensure good condition and no evidence of erosion. • Repair or replace any damaged structural parts. • Stabilize any eroded areas. • Ensure that flow is not bypassing the facility. • Ensure that no noticeable odors are detected outside the facility.	Annually
• If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till or otherwise cultivate the surface, and replace media with sand meeting the design specifications. • Replace any filter fabric that has become clogged.	As needed

Additional Maintenance Considerations and Requirements

- A record should be kept of the dewatering time for a sand filter to determine if maintenance is necessary.
- When the filtering capacity of the sand filter facility diminishes substantially (i.e., when water ponds on the surface of the filter bed for more than 48 hours), then the top layers of the filter media (topsoil and 2 to 3 inches of sand) will need to be removed and replaced. This will typically need to be done every 3 to 5 years for low sediment applications, more often for areas of high sediment yield or high oil and grease.
- Removed sediment and media may usually be disposed of in a landfill.

Regular inspection and maintenance is critical to the effective operation of sand filter facilities as designed. Maintenance responsibility for a sand filter system should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

5.2.15.8 Example Schematics

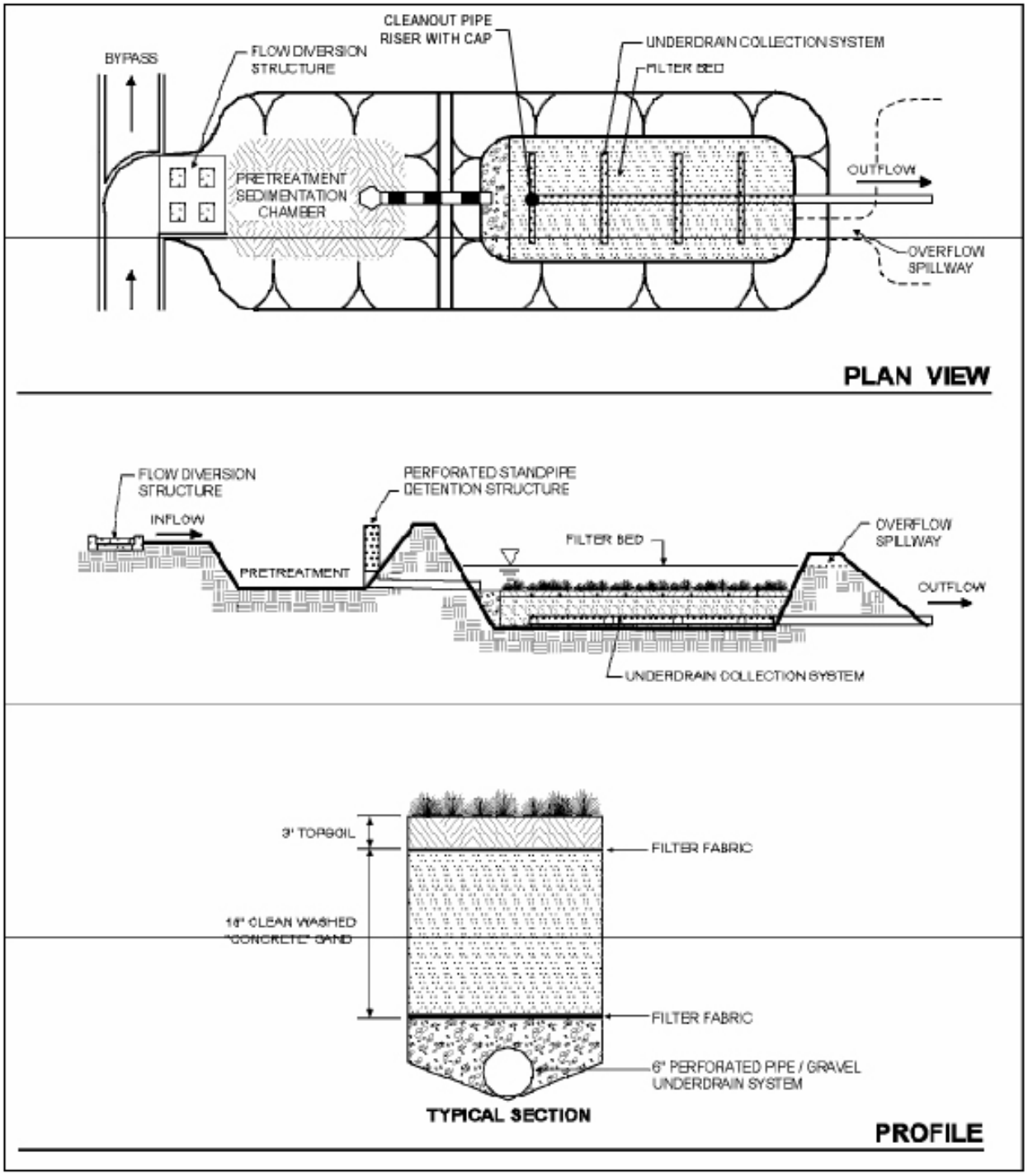

Figure 5.2.15-6 Schematic of Surface Sand Filter
(Source: Center for Watershed Protection)

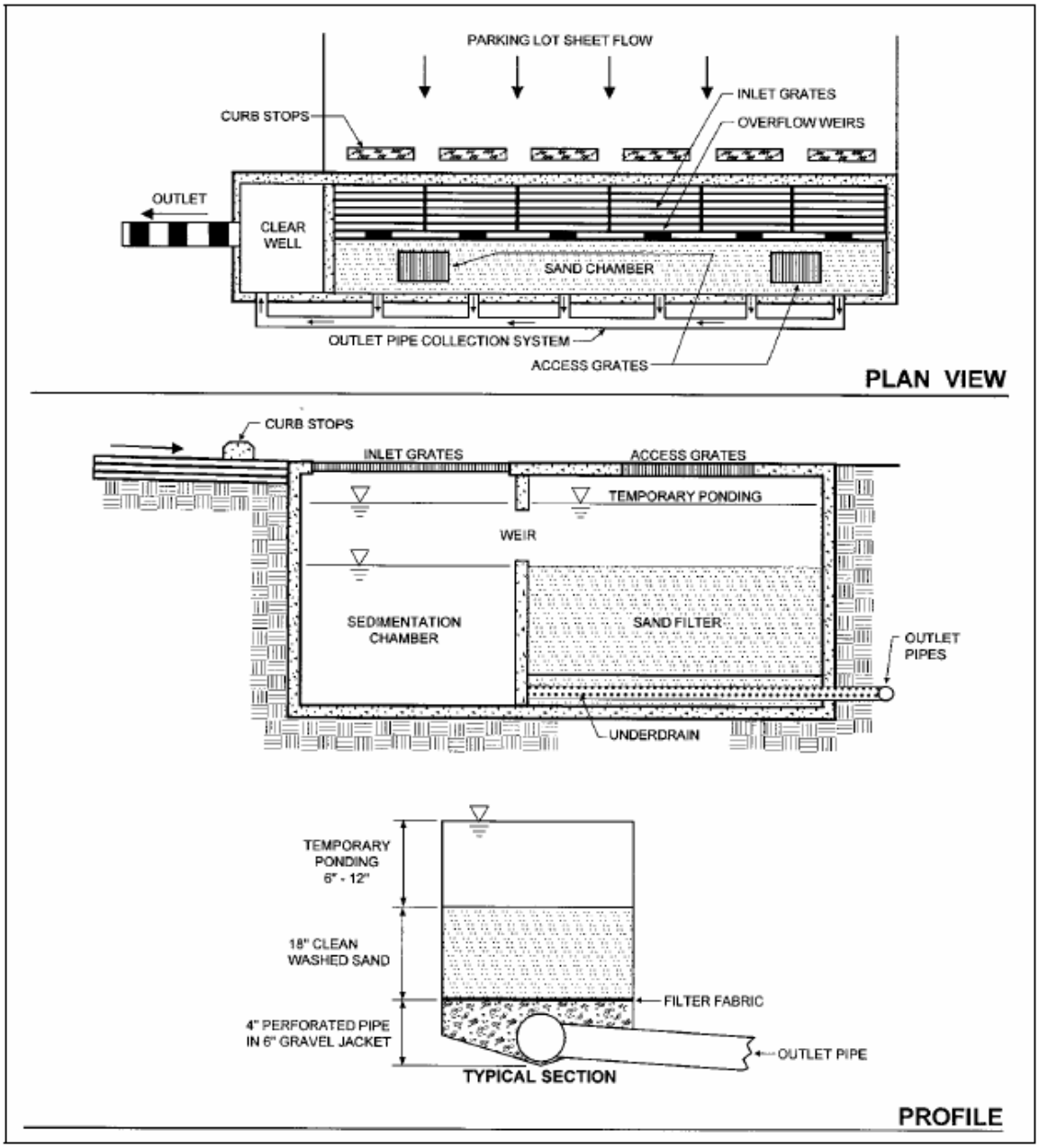

Figure 5.2.15-7 Schematic of Perimeter Sand Filter
(Source: Center for Watershed Protection)

5.2.15.9 Design Forms

Design Procedure Form: Sand Filters

PRELIMINARY HYDROLOGIC	
1a. Compute WQ_v volume requirements Copute Runoff Coefficient, R_v Compute WQ_v	R_v = _____ WQ_v = _____ acre-ft
1b. Compute SP_v Copute average release rate Compute Q_p (100-year detention volume required) Compute (as necessary) Q_f	SP_v = _____ acre-ft release rate = _____ cfs Q_p = _____ acre-ft Q_f = _____ cfs
SAND FILTER DESIGN	
2. Is the use of a sand fiilter appropriate?	Low Point in development area = _____ Low Point at stream invert = _____ Total available head = _____ Average depth, h_f = _____ ***See subsections 5.2.15.4 and 5.2.15.5-A***
3. Confirm localdesign criteria and applicability.	***See subsection 5.2.15.5-J***
4. Compute WQ_v peak discharge (Q_{vq}) Compute Curve Number Compute Time of Concentration t_c Compute Q_{wq}	CN = _____ t_c = _____ hour Q_{wq} = _____ cfs
5. Size flow diversion structure Low flow orifice - Orifice equation Overflow weir - Weir equation	A = _____ ft^2 diameter = _____ in Length = _____ ft
6. Size filtration bed chamber Compute area from Darcy's Law Using length to width (2:1) ratio	A_f = _____ ft^2 L = _____ ft W = _____ ft
7. Size seidmentation chamber Compute area from Camp-Hazen equation Given W from step 5, compute Length	A_f = _____ ft^2 L = _____ ft
8. Compute V_{min}	V_{min} = _____ ft^3
9. Compute volume within practice	
<u>Surface sand filter</u> Volume within filter bed Temporary storage above filter bed Sedimentation chamber (remaining volume) Height in sedimentation chamber Perforated stand pipe - Orifice equation	V_f = _____ ft^3 $V_{s\text{-}temp}$ = _____ ft^3 V_s = _____ ft^3 h_s = _____ ft A = _____ ft^2 diameter = _____ in
<u>Perimeter sand filter</u> Compute volume in filter bed Compute wet pool storage Compute temporary storage	V_f = _____ ft^3 V_w = _____ ft^3 V_{temp} = _____ ft^3 h_{temp} = _____ ft
10. Compute overflow weir sizes Compute overflow - Orifice equation Weir from sedimentation chamber - Weir equation Weir from filtration chamber - Weir equation	Q = _____ cfs Length = _____ ft Length = _____ ft

Notes:

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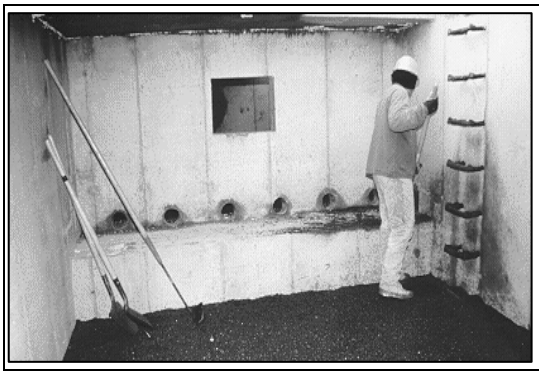
5.2.16 Underground Sand Filter

Limited Application
Structural Storm Water Control



Description: Design variant of the sand filter located in an underground vault.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for space-limited applications

ADVANTAGES / BENEFITS:

- High pollutant removal capability
- High removal rates for sediment, BOD, and fecal coliform bacteria
- Precast concrete shells available, which decrease construction costs

DISADVANTAGES / LIMITATIONS:

- High maintenance requirements

MAINTENANCE REQUIREMENTS:

- Filter may require more frequent maintenance than most of the other storm water controls

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/25%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
40%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Hotspot areas*

L=Low **M**=Moderate **H**=High

5.2.16.1 General Description

The underground sand filter is a design variant of the sand filter located in an underground vault designed for high-density land use or ultra-urban applications where there is not enough space for a surface sand filter or other structural storm water controls.

The underground sand filter is a three-chamber system. The initial chamber is a sedimentation (pretreatment) chamber that temporarily stores runoff and utilizes a wet pool to capture sediment. The sedimentation chamber is connected to the sand filter chamber by a submerged wall that protects the filter bed from oil and trash. The filter bed is 18 to 24 inches deep and may have a protective screen of gravel or permeable geotextile to limit clogging. The sand filter chamber also includes an underdrain system with inspection and clean out wells. Perforated drain pipes under the sand filter bed extend into a third chamber that collects filtered runoff. Flows beyond the filter capacity are diverted through an overflow weir.

Due to its location below the surface, underground sand filters have a high maintenance burden and should only be used where adequate inspection and maintenance can be ensured.

5.2.16.2 Pollutant Removal Capabilities

Underground sand filter pollutant removal rates are similar to those for surface and perimeter sand filters (see subsection 5.2.15, *Sand Filters*).

5.2.16.3 Design Criteria and Specifications

- Underground sand filters are typically used on highly impervious sites of 1 acre or less. The maximum drainage area that should be treated by an underground sand filter is 5 acres.
- Underground sand filters are typically constructed on-line, but can be constructed off-line. For off-line construction, the overflow between the second and third chambers is not included.
- The underground vault should be tested for water tightness prior to placement of filter layers.
- Adequate maintenance access must be provided to the sedimentation and filter bed chambers.
- Compute the minimum wet pool volume required in the sedimentation chamber as:

  $$V_w = A_s * 3 \text{ feet minimum}$$

- Consult the design criteria for the perimeter sand filter (see Section 5.2.15) for the rest of the underground filter sizing and design steps.

5.3.4.4 Inspection and Maintenance Requirements

Table 5.2.16-1 Typical Maintenance Activities for Underground Sand Filters
(Source: CWP, 1996)

Activity	Schedule
• Monitor water level in sand filter chamber.	Quarterly and following large storm events
• Sedimentation chamber should be cleaned out when the sediment depth reaches 12 inches.	As needed
• Remove accumulated oil and floatables in sedimentation chamber.	As needed, (typically every 6 months)

As a variant, organic material may be used instead of the sand media in the underground filter. Organic material has a higher cation exchange capacity and may remove more metals and other charged pollutants. Additional inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see subsection 5.2.15)

5.2.16.5 Example Schematic

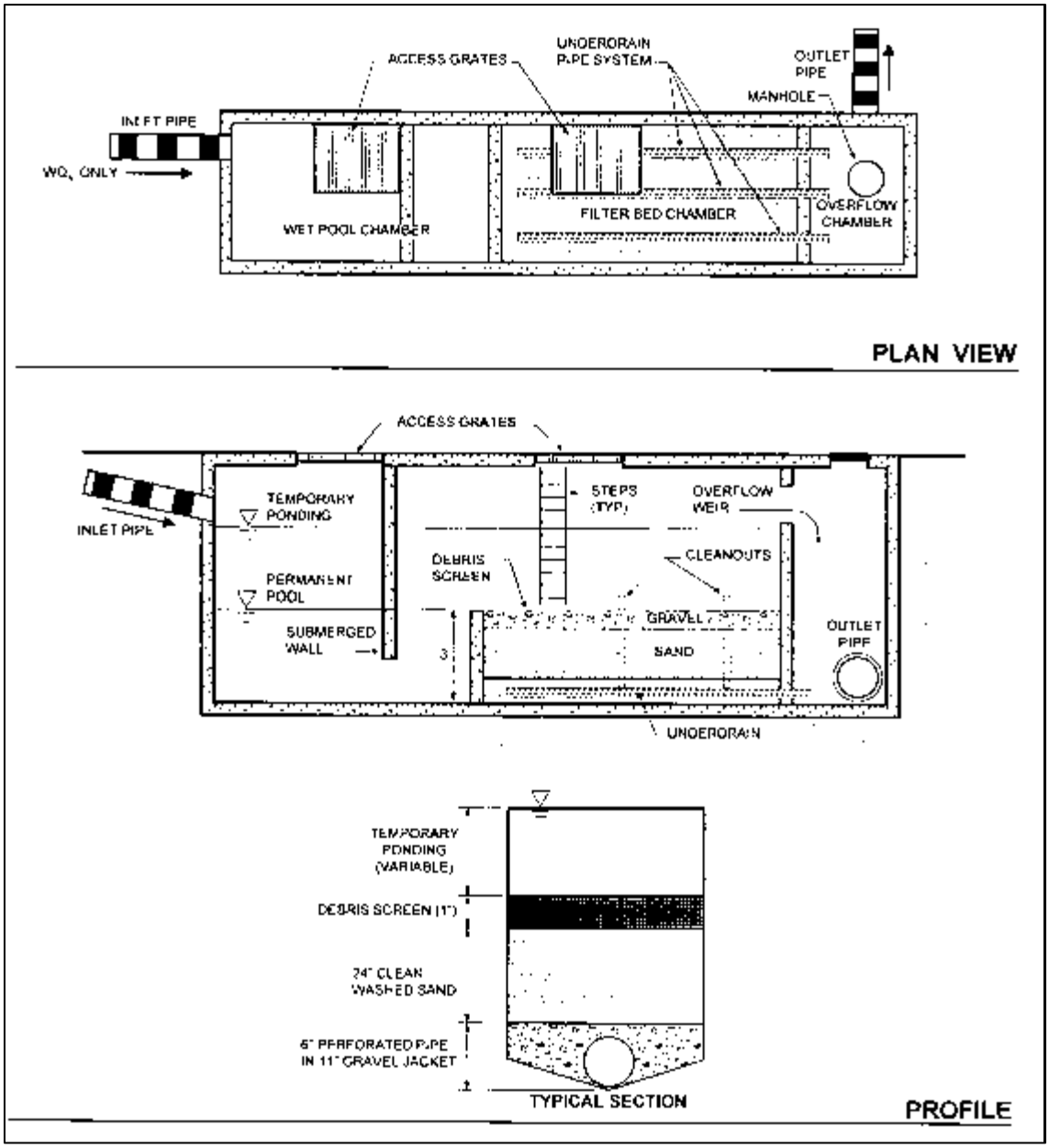


Figure 5.2.16-1 Schematic of Underground Sand Filter
(Source: Center for Watershed Protection)

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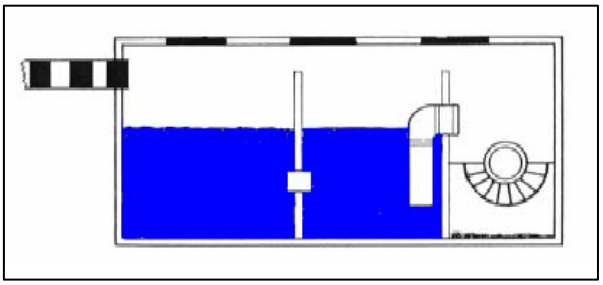
5.2.17 Gravity (Oil-Grit) Separator

Structural Storm Water Control



Description: Hydrodynamic separation device designed to remove settleable solids, oil and grease, debris, and floatables from storm water runoff through gravitational settling and trapping of pollutants.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for hotspot, space-limited, or pretreatment applications
- Intended for the removal of settleable solids (grit and sediment) and floatable matter, including oil and grease
- Performance dependent on design and frequency of inspection and cleanout of unit

DISADVANTAGES / LIMITATIONS:

- Cannot alone achieve the 80% TSS removal target
- Limited performance data
- Dissolved pollutants are not effectively removed

MAINTENANCE REQUIREMENTS:

- Frequent maintenance required

POLLUTANT REMOVAL

40%	**Total Suspended Solids**
5/5%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
No data	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [S] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *1 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Hotspot areas*
- *Pretreatment*

L=Low **M**=Moderate **H**=High

5.2.17.1 General Description

Gravity separators (also known as oil-grit separators) are hydrodynamic separation devices that are designed to remove grit and heavy sediments, oil and grease, debris, and floatable matter from storm water runoff through gravitational settling and trapping. Gravity separator units contain a permanent pool of water and typically consist of an inlet chamber, separation/storage chamber, a bypass chamber, and an access port for maintenance purposes. Runoff enters the inlet chamber where heavy sediments and

solids drop out. The flow moves into the main gravity separation chamber, where further settling of suspended solids takes place. Oil and grease are skimmed and stored in a waste oil storage compartment for future removal. After moving into the outlet chamber, the clarified runoff is then discharged.

The performance of these systems is based primarily on the relatively low solubility of petroleum products in water and the difference between the specific gravity of water and the specific gravities of petroleum compounds. Gravity separators are not designed to separate other products such as solvents, detergents, or dissolved pollutants. The typical gravity separator unit may be enhanced with a pretreatment swirl concentrator chamber, oil draw-off devices that continuously remove the accumulated light liquids, and flow control valves regulating the flow rate into the unit.

Gravity separators are best used in commercial, industrial, and transportation land uses and are intended primarily as a pretreatment measure for high-density or ultra urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. However, gravity separators cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols, and alcohols.

Since re-suspension of accumulated sediments is possible during heavy storm events, gravity separator units are typically installed off-line. Gravity separators are available as prefabricated proprietary systems from a number of different commercial vendors.

5.2.17.2 Pollutant Removal Capabilities

Testing of gravity separators has shown that they can remove between 40 and 50% of the TSS loading when used in an off-line configuration (Curran, 1996 and Henry, 1999). Gravity separators also provide removal of debris, hydrocarbons, trash and other floatables. They provide only minimal removal of nutrients and organic matter.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- **Total Suspended Solids – 40%**
- **Total Phosphorus – 5%**
- **Total Nitrogen – 5%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – insufficient data**

Actual field testing data and pollutant removal rates from an independent source should be obtained before using a proprietary gravity separator system.

5.2.17.3 Design Criteria and Specifications

- The use of gravity (oil-grit) separators should be limited to the following applications:
  - Pretreatment for other structural storm water controls
  - High-density, ultra urban or other space-limited development sites
  - Hotspot areas where the control of grit, floatables, and/or oil and grease are required
- Gravity separators are typically used for areas less than 5 acres. It is recommended that the contributing area to any individual gravity separator be limited to 1 acre or less of impervious cover.
- Gravity separator systems can be installed in almost any soil or terrain. Since these devices are underground, appearance is not an issue and public safety risks are low.

- Gravity separators are rate-based devices. This contrasts with most other storm water structural controls, which are sized based on capturing and treating a specific volume.
- Gravity separator units are typically designed to bypass runoff flows in excess of the design flow rate. Some designs have built-in high flow bypass mechanisms. Other designs require a diversion structure or flow splitter ahead of the device in the drainage system. An adequate outfall must be provided.
- The separation chamber should provide for three separate storage volumes:
  - a A volume for separated oil storage at the top of the chamber
  - b A volume for settleable solids accumulation at the bottom of the chamber
  - c A volume required to give adequate flow-through detention time for separation of oil and sediment from the storm water flow
- The total wet storage of the gravity separator unit should be at least 400 cubic feet per contributing impervious acre.
- The minimum depth of the permanent pools should be 4 feet.
- Horizontal velocity through the separation chamber should be 1 to 3 ft/min or less. No velocities in the device should exceed the entrance velocity.
- A trash rack should be included in the design to capture floating debris, preferably near the inlet chamber to prevent debris from becoming oil impregnated.
- Ideally, a gravity separator design will provide an oil draw-off mechanism to a separate chamber or storage area.
- Adequate maintenance access to each chamber must be provided for inspection and cleanout of a gravity separator unit.
- Gravity separator units should be watertight to prevent possible groundwater contamination.
- The design criteria and specifications of a proprietary gravity separator unit should be obtained from the manufacturer.

5.2.17.4 Inspection and Maintenance Requirements

Table 5.2.17-1 Typical Maintenance Activities for Gravity Separators

Activity	Schedule
• Inspect the gravity separator unit for structural problems, accumulated pollutants, and mosquito larvae.	Regularly (quarterly)
• Clean out sediment, oil and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As Needed

Additional Maintenance Considerations and Requirements

- Additional maintenance requirements for a proprietary system should be obtained from the manufacturer.
- Failure to provide adequate inspection and maintenance can result in the re-suspension of accumulated solids. Frequency of inspection and maintenance is dependent on land use, climatological conditions, and the design of gravity separator.
- Proper disposal of oil, solids, and floatables removed from the gravity separator must be ensured.
- If mosquito larvae are present in the unit, treat with larvacide. (See sub-section 5.2.18.4)

5.2.17.5 Example Schematic

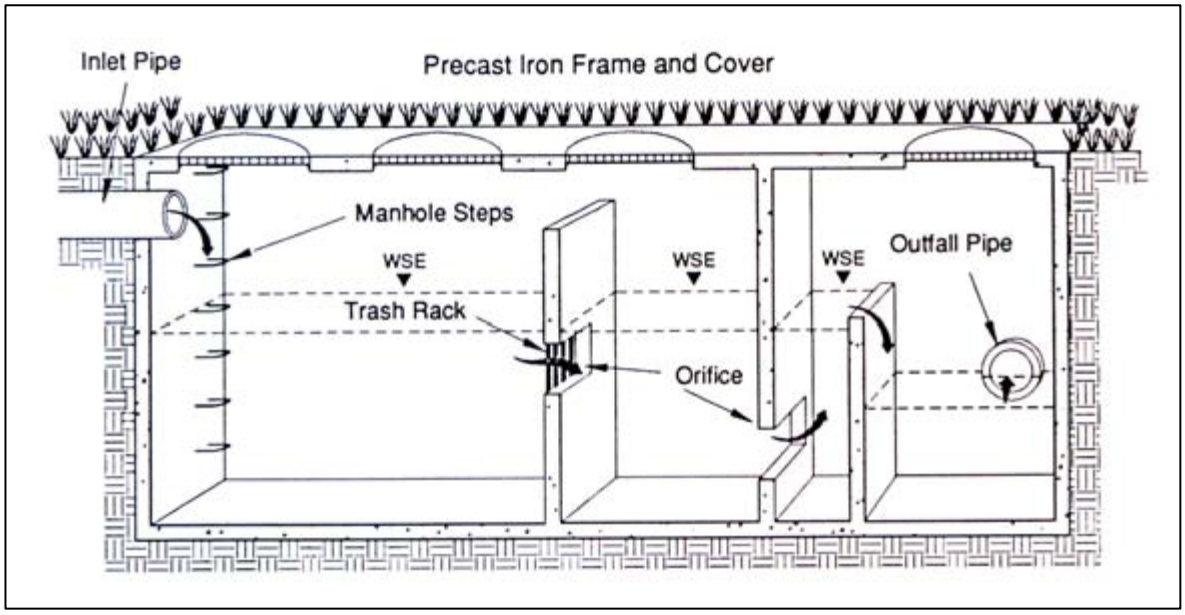
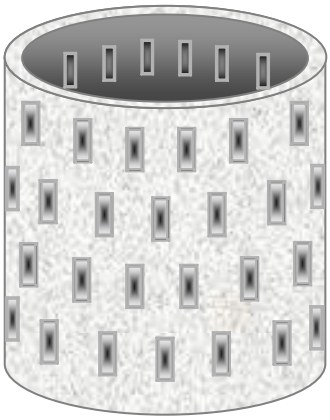


Figure 5.2.17-1 Schematic of an Example Gravity (Oil-Grit) Separator
(Source: NVRC, 1992[1])

5.2.18 Downspout Drywell

Structural Storm Water Control

Description: Drywells are essentially perforated manholes, but they can be manufactured in various sizes. Located underground, they allow storm water infiltration even in highly urbanized areas. They should be used in conjunction with some type of pretreatment devices where there are minimal risks of groundwater contamination.



KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for space-limited applications
- Like other infiltration devices, drywells should not be used for storm water containing high sediment loads to minimize clogging

ADVANTAGES / BENEFITS:

- Filtration provides pollutant removal capability in adjacent soil
- Decreases peak flow rates

DISADVANTAGES / LIMITATIONS:

- Subsurface structure considered an injection well and may require special permit

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
90%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **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: *No restrictions*

Soils: *Pervious soils required (0.5 in/hr or greater)*

L=Low **M**=Moderate **H**=High

5.2.18.1 General Description

Drywells are infiltration devices that have historically been used to dispose of excess runoff without extensive infrastructure. Its minimal land requirements allow it to be used in highly urbanized areas. Drywells used for storm water disposal are considered Class V injection devices by the EPA and fall under the Texas UIC program. Concerns about contaminating aquifers limit their application to "clean" runoff, such as roofdrains, and require pretreatment devices to remove sediments and other pollutants.

Drywells should not be used in areas near drinking water wells, with industrial land use, with high groundwater tables, a substrate of fractured rock, or slow-draining soils. Drywell design should be overseen by a licensed engineer.

5.2.18.2 Pollutant Removal Capabilities

Pollutant removal is similar to infiltration trenches (see Section 5.2.17), but care should be taken to avoid clogging with sediments.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 60%**
- **Total Nitrogen – 60%**
- **Fecal Coliform – 90%**
- **Heavy Metals – 90%**

5.2.18.3 Design Criteria and Specifications

- The drywell should be located at least 5 feet from the nearest property line and 10 feet away from an occupied building.
- Drywells shall be located at least 200 feet from the tops of slopes more than 10 feet high and steeper than 2h:1v.
- The drywell shall be excavated in native soil, uncompacted by heavy equipment.
- A qualified professional shall conduct infiltration testing. The surrounding soil should have a minimum infiltration rate of 0.5 inches per hour.
- The drywell shall be surrounded by a 12 inch thick layer of ¾" to 2 ½" round rock.
- There should be at least four feet between the bottom of the drywell and the seasonal high ground water table or bedrock.
- A pretreatment device should be installed upstream of the drywell to remove sediments and other pollutants.
- The drywell shall be sized in accordance with the simplified sizing criteria.
- The drywell should not be located next to trees, since roots may penetrate drywell and clog it.
- Access should be provided for drywell maintenance via a secured manhole or cleanout.

5.2.18.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for drywells are designed to maintain an adequate drainage rate through the drywell, while avoiding groundwater contamination.

Table 5.2.18-1 Typical Maintenance Activities for Drywells

Activity	Schedule
• Ensure that inflow is unimpeded. Clean out accumulated sediment/ debris and dispose of properly.	Quarterly and within 48 hours of major storms
• Inspect pretreatment device and clean if necessary. Cleaning shall be done without the used of detergents or solvents.	As needed, based on minimum annual inspection
• Inspect area surrounding the drywell for waterlogged soils at surface, indicating drywell failure. Clogged drywells must be replaced.	Inspect between 24 - 48 hours after major storms
• Pest control measures shall be taken if rodents or mosquitoes are found to be present. Holes in the ground around the drywell shall be filled and a low toxicity mosquito larvacide, such as Bacillus thuringiensis (Bti), Bacillus Sphearicus (Bsph) or Methoprene (insect growth regulator) applied by a licensed individual, if necessary.	As needed

5.2.18.5 Example Schematic

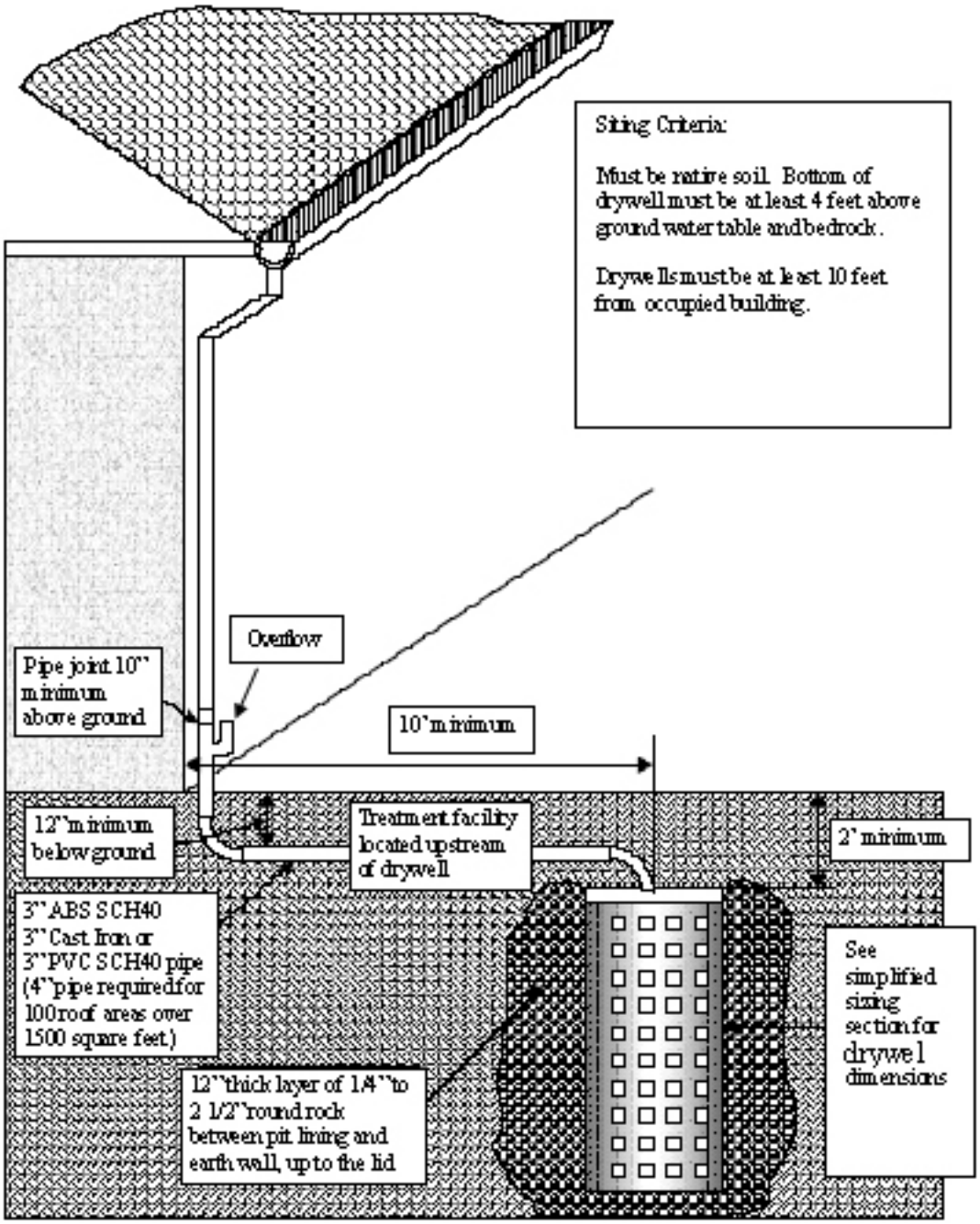


Figure 5.2.18-1 Schematic of Drywell System
(Source: City of Portland, Oregon)

5.2.19 Infiltration Trench

Structural Storm Water Control

Description: Excavated trench filled with stone aggregate used to capture and allow infiltration of storm water runoff into the surrounding soils from the bottom and sides of the trench.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Soil infiltration rate of 0.5 in/hr or greater required
- Excavated trench (3 to 8 foot depth) filled with stone media (1.5- to 2.5-inch diameter); pea gravel, and sand filter layers
- A sediment forebay and grass channel, or equivalent upstream pretreatment, must be provided
- Observation well to monitor percolation

ADVANTAGES / BENEFITS:

- Provides for groundwater recharge
- Good for small sites with porous soils

DISADVANTAGES / LIMITATIONS:

- Potential for groundwater contamination
- High clogging potential; should not be used on sites with fine-particled soils (clays or silts) in drainage area
- Significant setback requirements
- Restrictions in karst areas
- Geotechnical testing required, two borings per facility

MAINTENANCE REQUIREMENTS:

- Inspect for clogging
- Remove sediment from forebay
- Replace pea gravel layer as needed

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
90%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E. Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **Water Quality Protection**
- [S] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [M] **Land Requirement**
- [H] **Capital Cost**
- [H] **Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *Pervious soils required (0.5 in/hr or greater)*

Other Considerations:

- *Must not be placed under pavement or concrete*

L=Low **M**=Moderate **H**=High

5.2.19.1 General Description

Infiltration trenches are excavations typically filled with stone to create an underground reservoir for storm water runoff (see Figure 5.2.19-1). This runoff volume gradually filtrates through the bottom and sides of the trench into the subsoil over a 2-day period and eventually reaches the water table. By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site and can recharge groundwater and preserve baseflow. Due to this fact, infiltration systems are limited to areas with highly porous soils where the water table and/or bedrock are located well below the bottom of the trench. In addition, infiltration trenches must be carefully sited to avoid the potential of groundwater contamination.

Infiltration trenches are not intended to trap sediment and must always be designed with a sediment forebay and grass channel or filter strip, or other appropriate pretreatment measures to prevent clogging and failure. Due to their high potential for failure, these facilities must only be considered for sites where upstream sediment control can be ensured.

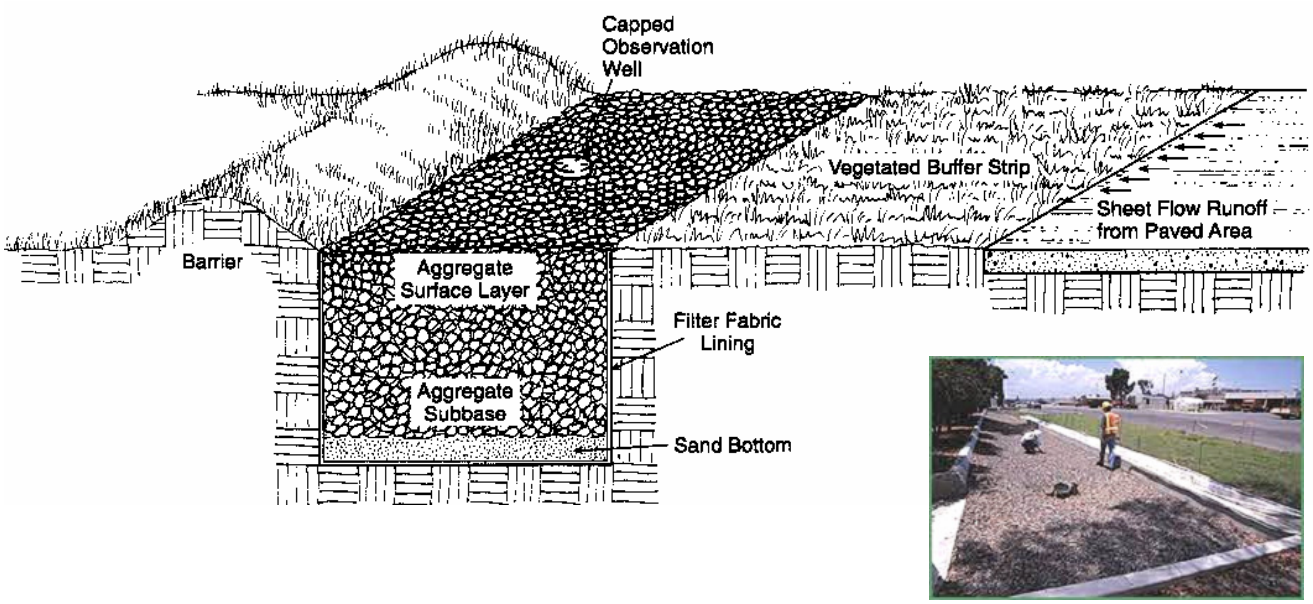


Figure 5.2.19-1 Infiltration Trench Example

5.2.19.2 Storm Water Management Suitability

Infiltration trenches are designed primarily for storm water quality, i.e. the removal of storm water pollutants. However, they can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the streambank protection volume (SP_v) in addition to WQ_v. An infiltration trench will need to be used in conjunction with another structural control to provide flood control, if required.

Water Quality Protection

Using the natural filtering properties of soil, infiltration trenches can remove a wide variety of pollutants from storm water through sorption, precipitation, filtering, and bacterial and chemical degradation. Sediment load and other suspended solids are removed from runoff by pretreatment measures in the facility that treats flows before they reach the trench surface.

Section 5.2.19.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

For smaller sites, an infiltration trench may be designed to capture and infiltrate the entire streambank protection volume SP_v in either an off- or on-line configuration. For larger sites, or where only the WQ_v is diverted to the trench, another structural control must be used to provide SP_v extended detention.

Flood Control

Infiltration trench facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

The volume of runoff removed and treated by the infiltration trench may be taken in the on-site and/or downstream flood control calculations (see Section 5.1).

5.2.19.3 Pollutant Removal Capabilities

An infiltration trench is presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed infiltration trenches can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 60%**
- **Total Nitrogen – 60%**
- **Fecal Coliform – 90%**
- **Heavy Metals – 90%**

For additional information and data on pollutant removal capabilities for infiltration trenches, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Storm Water Best Management Practices (BMP) Database at www.bmpdatabase.org

5.2.19.4 Application and Site Feasibility Criteria

Infiltration trenches are generally suited for medium-to-high density residential, commercial, and institutional developments where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and the water table is low enough to prevent groundwater contamination. They are applicable primarily for impervious areas where there are not high levels of fine particulates (clay/silt soils) in the runoff and should only be considered for sites where the sediment load is relatively low.

Infiltration trenches can either be used to capture sheet flow from a drainage area or function as an off-line device. Due to the relatively narrow shape, infiltration trenches can be adapted to many different types of sites and can be utilized in retrofit situations. Unlike some other structural storm water controls, they can easily fit into the margin, perimeter, or other unused areas of developed sites.

To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Infiltration trenches should not be used for manufacturing and industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals. In

addition, infiltration should not be considered for areas with a high pesticide concentration. Infiltration trenches are also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements.

The following criteria should be evaluated to ensure the suitability of an infiltration trench for meeting storm water management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Storm Water Control – NO

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum
- Space Required – Will vary depending on the depth of the facility
- Site Slope – No more than 6% slope (for pre-construction facility footprint)
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 1 foot
- Minimum Depth to Water Table – 4 feet recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table
- Soils – Infiltration rate greater than 0.5 inches per hour required (typically hydrologic group "A", some group "B" soils)

Other Constraints / Considerations

- Aquifer Protection – No hotspot runoff allowed; meet setback requirements in design criteria

5.2.19.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of an infiltration trench facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. Location and Siting

- To be suitable for infiltration, underlying soils should have an infiltration rate (f_c) of 0.5 inches per hour or greater, as initially determined from NRCS soil textural classification and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5,000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils.
- Infiltration trenches should have a contributing drainage area of 5 acres or less.
- Soils on the drainage area tributary to an infiltration trench should have a clay content of less than 20% and a silt/clay content of less than 40% to prevent clogging and failure.
- There should be at least 4 feet between the bottom of the infiltration trench and the elevation of the seasonally high water table.
- Clay lenses, bedrock or other restrictive layers below the bottom of the trench will reduce infiltration rates unless excavated.

- Minimum setback requirements for infiltration trench facilities (when not specified by local ordinance or criteria):
  - From a property line – 10 feet
  - From a building foundation – 25 feet
  - From a private well – 100 feet
  - From a public water supply well – 1,200 feet
  - From a septic system tank/leach field – 100 feet
  - From surface waters – 100 feet
  - From surface drinking water sources – 400 feet (100 feet for a tributary)
- When used in an off-line configuration, the water quality protection volume (WQ_v) is diverted to the infiltration trench through the use of a flow splitter. Storm water flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.
- To reduce the potential for costly maintenance and/or system reconstruction, it is strongly recommended that the trench be located in an open or lawn area, with the top of the structure as close to the ground surface as possible. Infiltration trenches shall not be located beneath paved surfaces, such as parking lots.
- Infiltration trenches are designed for intermittent flow and must be allowed to drain and allow re-aeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

B. GENERAL DESIGN

- A well-designed infiltration trench consists of:
  - Excavated shallow trench backfilled with sand, coarse stone, and pea gravel, and lined with a filter fabric
  - Appropriate pretreatment measures
  - One or more observation wells to show how quickly the trench dewaters or to determine if the device is clogged

Figure 5.2.19-2 provides a plan view and profile schematic for the design of an off-line infiltration trench facility. An example of an on-line infiltration trench is shown in Figure 5.2.19-1.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- The required trench storage volume is equal to the water quality protection volume (WQ_v). For smaller sites, an infiltration trench can be designed with a larger storage volume to include the streambank protection volume (SP_v).
- A trench must be designed to fully dewater the entire WQ_v within 24 to 48 hours after a rainfall event. The slowest infiltration rate obtained from tests performed at the site should be used in the design calculations.
- Trench depths should be between 3 and 8 feet, to provide for easier maintenance. The width of a trench must be less than 25 feet.
- Broader, shallow trenches reduce the risk of clogging by spreading the flow over a larger area for infiltration.
- The surface area required is calculated based on the trench depth, soil infiltration rate, aggregate void space, and fill time (assume a fill time of 2 hours for most designs).
- The bottom slope of a trench should be flat across its length and width to evenly distribute flows, encourage uniform infiltration through the bottom, and reduce the risk of clogging.

- The stone aggregate used in the trench should be washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40%. Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations, unless aggregate specific data exist.
- A 6-inch layer of clean, washed sand is placed on the bottom of the trench to encourage drainage and prevent compaction of the native soil while the stone aggregate is added.
- The infiltration trench is lined on the sides and top by an appropriate geotextile filter fabric that prevents soil piping but has greater permeability than the parent soil. The top layer of filter fabric is located 2 to 6 inches from the top of the trench and serves to prevent sediment from passing into the stone aggregate. Since this top layer serves as a sediment barrier, it will need to be replaced more frequently and must be readily separated from the side sections.
- The top surface of the infiltration trench above the filter fabric is typically covered with pea gravel. The pea gravel layer improves sediment filtering and maximizes the pollutant removal in the top of the trench. In addition, it can easily be removed and replaced should the device begin to clog. Alternatively, the trench can be covered with permeable topsoil and planted with grass in a landscaped area.
- An observation well must be installed in every infiltration trench and should consist of a perforated PVC pipe, 4 to 6 inches in diameter, extending to the bottom of the trench (see Figure 5.2.19-3 for an observation well detail). The observation well will show the rate of dewatering after a storm, as well as provide a means of determining sediment levels at the bottom and when the filter fabric at the top is clogged and maintenance is needed. It should be installed along the centerline of the structure, flush with the ground elevation of the trench. A visible floating marker should be provided to indicate the water level. The top of the well should be capped and locked to discourage vandalism and tampering.
- The trench excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench so as not to jeopardize the stability of the trench sidewalls. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and should be scarified prior to placement of sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction and should be constructed after upstream areas have been stabilized.

D. PRETREATMENT / INLETS

- Pretreatment facilities **must always** be used in conjunction with an infiltration trench to prevent clogging and failure.
- For a trench receiving sheet flow from an adjacent drainage area, the pretreatment system should consist of a vegetated filter strip with a minimum 25-foot length. A vegetated buffer strip around the entire trench is required if the facility is receiving runoff from both directions. If the infiltration rate for the underlying soils is greater than 2 inches per hour, 50% of the WQ_v should be pretreated by another method prior to reaching the infiltration trench.
- For an off-line configuration, pretreatment should consist of a sediment forebay, vault, plunge pool, or similar sedimentation chamber (with energy dissipaters) sized to 25% of the water quality protection volume (WQ_v). Exit velocities from the pretreatment chamber must be nonerosive for the 2-year design storm.

E. OUTLET STRUCTURES

- Outlet structures are not required for infiltration trenches.

F. EMERGENCY SPILLWAY

- Typically, for off-line designs, there is no need for an emergency spillway. However, a nonerosive overflow channel should be provided to pass safely flows that exceed the storage capacity of the trench to a stabilized downstream area or watercourse.

G. Maintenance Access

- Adequate access should be provided to an infiltration trench facility for inspection and maintenance.

H. Safety Features

- In general, infiltration trenches are not likely to pose a physical threat to the public and do not need to be fenced.

I. LANDSCAPING

- Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible. The trench may be covered with permeable topsoil and planted with grass in a landscaped area

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – No additional criteria
- High Relief – Maximum site slope of 6%
- Karst – Not suitable without adequate geotechnical testing

Special Downstream Watershed Considerations

No additional criteria

5.2.19.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Approach

Calculate the Water Quality Protection Volume (WQ_v), Streambank Protection Volume (SP_v), and the 100-Year Flood (Q_f).

Details on the *integrated* Design Approach are found in Section 1.2.

Step 2 Determine if the development site and conditions are appropriate for the use of an infiltration trench.

Consider the Application and Site Feasibility Criteria in subsections 5.2.19.4 and 5.2.19.5-A (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 5.2.19.5-J (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion (see subsection 2.1.7).

a Using WQ_v (or total volume to be infiltrated), compute CN
b Compute time of concentration using TR-55 method
c Determine appropriate unit peak discharge from time of concentration
d Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5 Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the infiltration trench.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6 Size infiltration trench

The area of the trench can be determined from the following equation:

$$A = \frac{WQ_v}{(nd + kT/12)}$$

where:

A	=	Surface Area
WQ_v	=	Water Quality Protection Volume (or total volume to be infiltrated)
n	=	porosity
d	=	trench depth (feet)
k	=	percolation (inches/hour)
T	=	Fill Time (time for the practice to fill with water), in hours

A porosity value *n* = 0.32 should be used.

All infiltration systems should be designed to fully dewater the entire WQ_v within 24 to 48 hours after the rainfall event.

A fill time T=2 hours can be used for most designs

See subsection 5.2.19.5-C (Physical Specifications/Geometry) for more specifications.

Step 7 Determine pretreatment volume and design pretreatment measures

Size pretreatment facility to treat 25% of the water quality protection volume (WQ_v) for off-line configurations.

See subsection 5.2.19.5-D (Pretreatment / Inlets) for more details.

Step 8 Design spillway(s)

Adequate storm water outfalls should be provided for the overflow exceeding the capacity of the trench, ensuring nonerosive velocities on the down-slope.

See Appendix D-4 for an Infiltration Trench Design Example

5.2.19.7 Inspection and Maintenance Requirements

Table 5.2.19-1 Typical Maintenance Activities for Infiltration Trenches

Activity	Schedule
• Ensure that contributing area, facility, and inlets are clear of debris. • Ensure that the contributing area is stabilized. • Remove sediment and oil/grease from pretreatment devices, as well as overflow structures. • Mow grass filter strips as necessary. Remove grass clippings.	Monthly
• Check observation wells following 3 days of dry weather. Failure to percolate within this time period indicates clogging. • Inspect pretreatment devices and diversion structures for sediment build-up and structural damage. • Remove trees that start to grow in the vicinity of the trench.	Semi-annual Inspection
• Replace pea gravel/topsoil and top surface filter fabric (when clogged).	As needed
• Perform total rehabilitation of the trench to maintain design storage capacity. • Excavate trench walls to expose clean soil.	Upon Failure

(Source: EPA, 1999)

Additional Maintenance Considerations and Requirements

- A record should be kept of the dewatering time of an infiltration trench to determine if maintenance is necessary.
- Removed sediment and media may usually be disposed of in a landfill.

Regular inspection and maintenance is critical to the effective operation of infiltration trench facilities as designed. Maintenance responsibility for an infiltration trench should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

5.2.19.8 Example Schematics

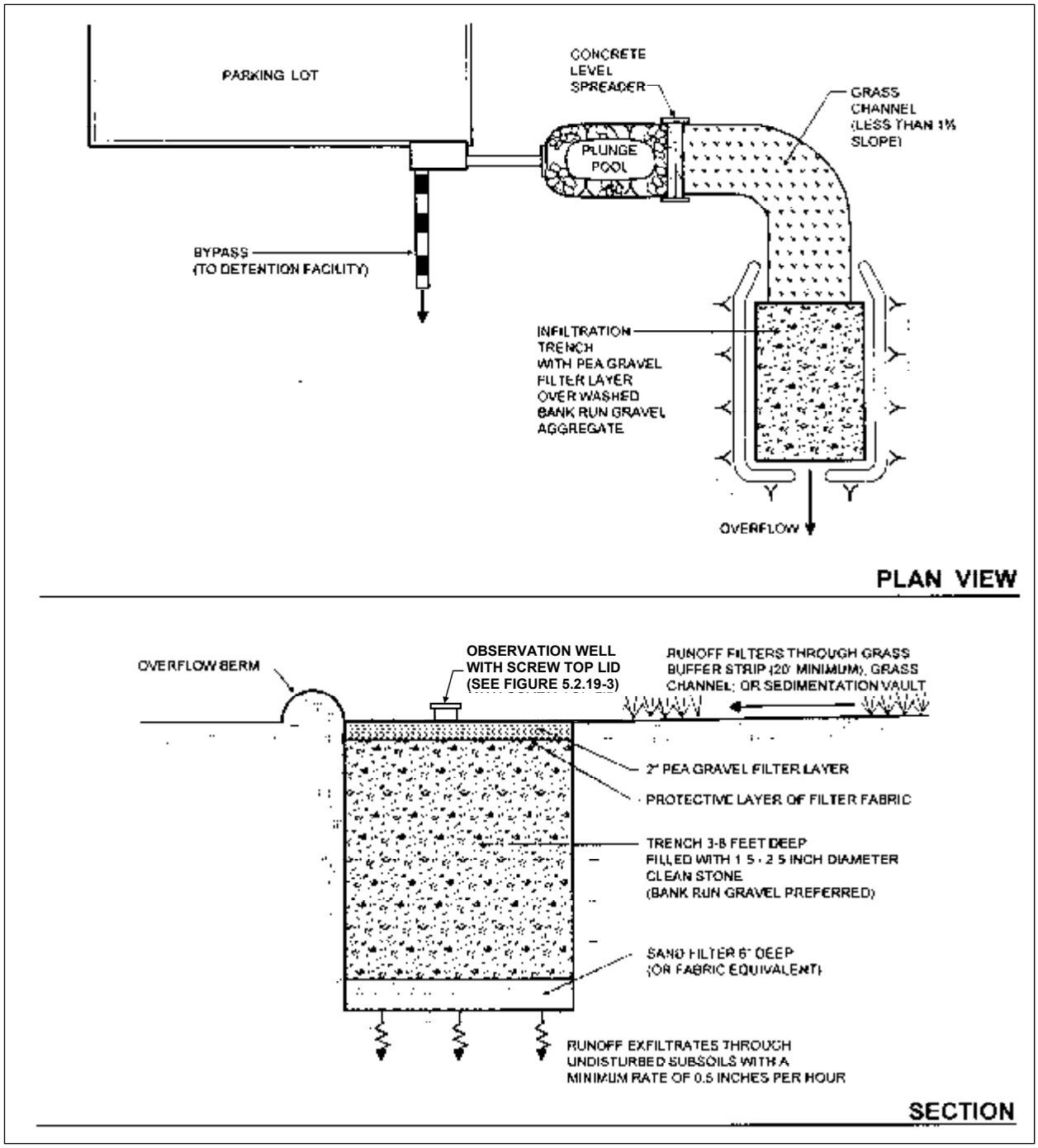


Figure 5.2.19-2 Schematic of Infiltration Trench
(Source: Center for Watershed Protection)

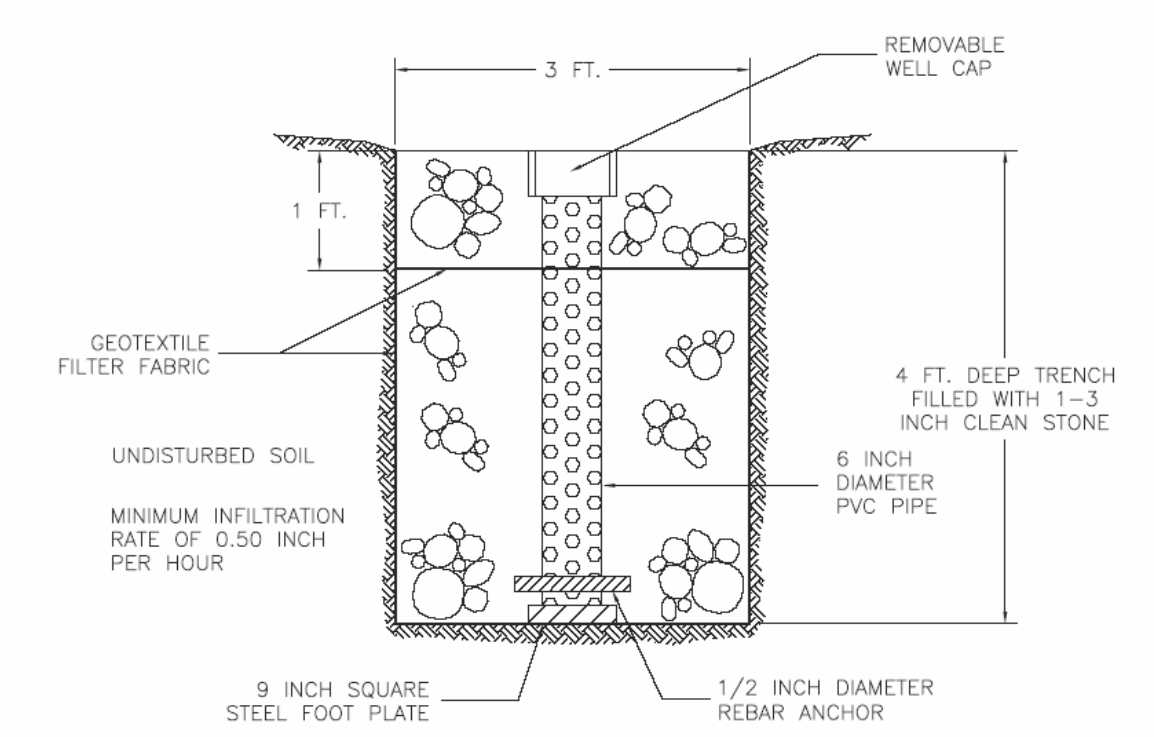


Figure 5.2.19-3 Observation Well Detail

- The aggregate material for the trench should consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches.
- The aggregate should be graded such that there will be few aggregates smaller than the selected size. For design purposes, void space for these aggregates may be assumed to be in the range of 30 to 40%.
- A 6-inch layer of clean, washed sand is placed on the bottom of the trench to encourage drainage and prevent compaction of the native soil, while the stone aggregate is added.
- The aggregate should be completely surrounded with an engineering filter fabric. If the trench has an aggregate surface, filter fabric should surround all of the aggregate fill material except for the top 1 foot.
- The observation well should consist of perforated PVC pipe, 4 to 6 inches diameter, located in the center of the structure, and be constructed flush with the ground elevation of the trench.
- The PVC pipe should have a factory attached cast iron or high impact to prevent rotation when removing the screw top lid.
- The screw top lid should be cast iron and clearly labeled as an observation well.

5.2.19.9 Design Forms

Design Procedure Form: Infiltration Trench

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements Compute Runoff Coefficient, R_v Compute WQv	R_v = _____ WQ_v = _____ acre-ft
1b. Compute $\mathbf{SP_v}$ Compute average release rate Compute Q_p (100-year detention volume required) Compute (as necessary) Q_f	$\mathbf{SP_v}$ = _____ acre-ft release rate = _____ cfs Q_p = _____ acre-ft Q_f = _____ cfs
INFILTRATION TRENCH DESIGN	
2. Is the use of a infiltration trench appropriate?	***See subsections 5.2.19.4 and 5.2.19.5 - A***
3. Confirm local design criteria and applicability.	***See subsection 5.2.19.5 - J***
4. Compute WQ_v peak discharge (Q_{wq}) Compute Curve Number Compute Time of Concentration t_c Compute Q_{wq}	CN = _____ t_c = _____ hour Q_{wq} = _____ cfs
5. Size infiltration trench Width must be less than 25 ft	Area = _____ ft^2 Width = _____ ft Length = _____ ft
6. Size the flow diversion structures Low flow orifice from orifice equation $Q = CA(2gh)^{0.5}$ Overflow weir from weir equation $Q = CLH^{3/2}$	A = _____ ft^2 diam. = _____ inch Length = _____ ft
7. Pretreatment volume (for offine designs) $Vol_{pre} = 0.25(WQ_v)$	Vol_{pre} = _____ ft^3
8. Design spillway(s)	

Notes: _____

5.2.20 Soakage Trench

Structural Storm Water Control

Description: Soakage trenches are a variation of infiltration trenches. Soakage trenches drain through a perforated pipe buried in gravel. They are used in highly impervious areas where conditions do not allow surface infiltration and where pollutant concentrations in runoff are minimal (i.e. non-industrial rooftops. They may be used in conjunction with other storm water devices, such as draining downspouts or planter boxes.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for space-limited applications
- Like other infiltration devices, soakage trenches should not be used for storm water containing high sediment loads to minimize clogging

ADVANTAGES / BENEFITS:

- Filtration provides pollutant removal capability
- Reservoir decreases peak flow rates

DISADVANTAGES / LIMITATIONS:

- Subsurface pipe considered an injection well and may require special permit

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
90%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **Water Quality Protection**
- [S] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max*

Soils: *Pervious soils required (0.5 in/hr or greater)*

L=Low **M**=Moderate **H**=High

5.2.20.1 General Description

Soakage trenches represent a variation of infiltration trench. Regular infiltration trenches drain from the surface, but in highly urbanized areas there is not often a suitable area available for this type of setup. Soakage trenches utilize a perforated pipe embedded within the trench, thereby minimizing the surface area required for the device. They can even be located under pavement.

Soakage trenches used for storm water disposal are considered Class V injection devices by the EPA and fall under the Texas UIC program.

5.2.20.2 Pollutant Removal Capabilities

Pollutant removal is similar to infiltration trenches (see Section 5.2.12), but care should be taken to avoid clogging with sediments.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 60%**
- **Total Nitrogen – 60%**
- **Fecal Coliform – 90%**
- **Heavy Metals – 90%**

5.2.20.3 Design Criteria and Specifications

- The soakage trench should be located at least 5 feet from the nearest property line and 10 feet away from an occupied building (they may be closer to other structures, such as a parking garage or other structures on piers.
- The trench shall be excavated in native soil, uncompacted by heavy equipment.
- The trench should be at least 3 feet deep and 2.5 feet wide as shown in Figure 5.2.20-1. The exact dimensions will be dependent on the drainage characteristics of the surrounding soils.
- There should be at least four feet between the bottom of the trench and the seasonal high ground water table.
- A silt trap or similar device may be installed upstream of the perforated pipe if pretreatment is needed prior to discharge.
- The bottom of the trench should be filled with at least 18 inches of medium sand meeting TxDOT Fine Aggregate Grade No 1 and covered with a layer of filter fabric.
- A minimum of six inches of ¾” – 2 ½” round or crushed rock shall be placed on top of the fabric covered sand base.
- Piping should be 3” diameter prior to the perforated drainage pipe, 4” if serving greater than 1500 square feet of roof.
- The perforated pipe shall be an approved leach field pipe with holes oriented downward. It shall be covered with filter fabric, with at least 12“ of backfill above the pipe.

5.2.20.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for soakage trenches are designed to maintain an adequate drainage rate through the trench, avoiding flooding.

Table 5.2.20-1 Typical Maintenance Activities for Soakage Trenches

Activity	Schedule
• Ensure that inflow is unimpeded.	Quarterly and within 48 hours of major storms
• Clean silt trap if it is more than 25% full of sediment	As needed, based on minimum annual inspection
• Inspect trench for waterlogged soils at surface.	Between 24 - 48 hours after major storms

5.2.20.5 Example Schematics

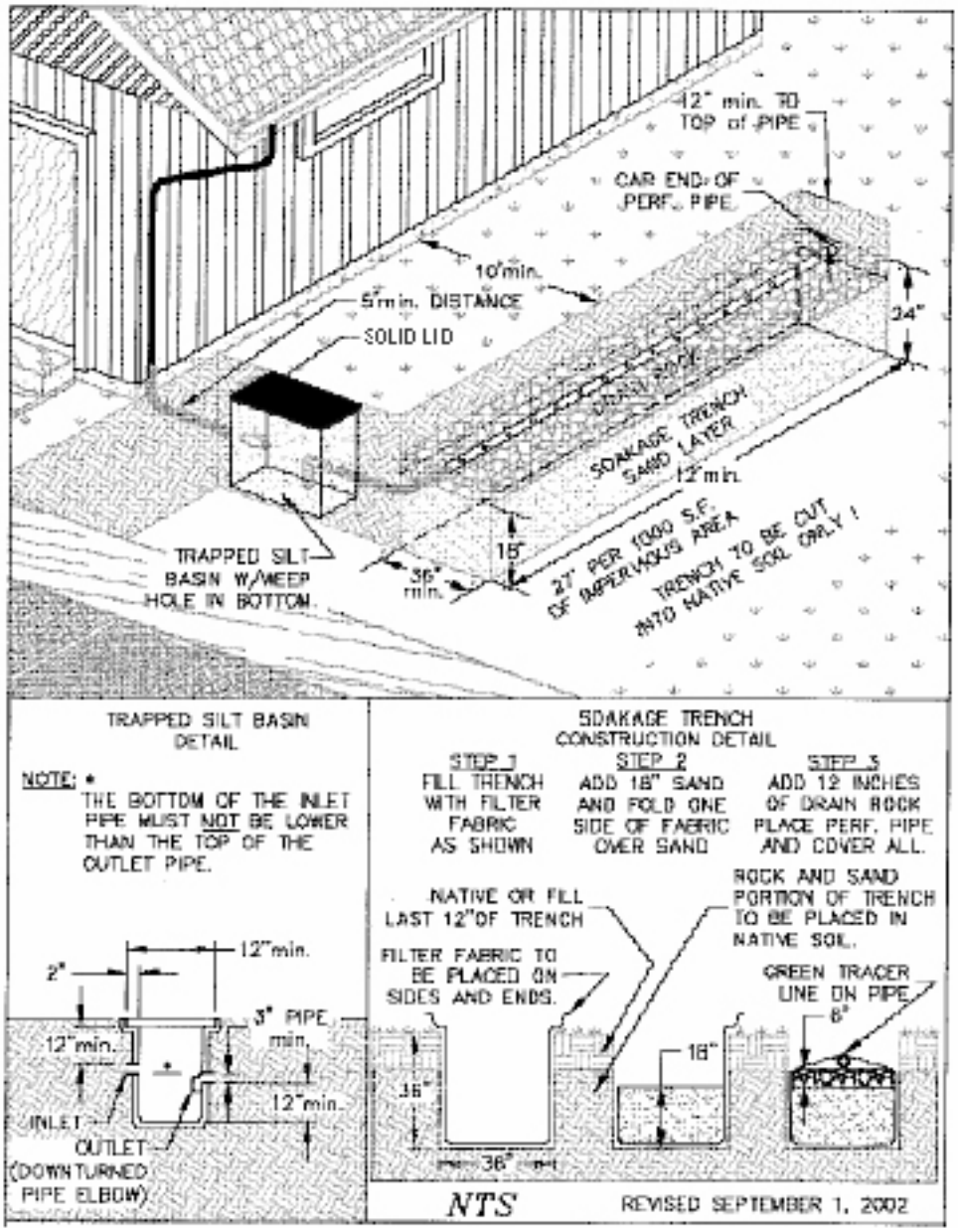


Figure 5.2.20-1 Schematic of a Soakage Trench
(Source: City of Portland, Oregon)

5.2.21 Storm Water Ponds

Storm Water Control

Description: Constructed storm water retention basin that has a permanent pool (or micropool). Runoff from each rain event is detained and treated in the pool primarily through settling and biological uptake mechanisms.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum contributing drainage area of 25 acres; 10 acres for extended detention micropool pond
- A sediment forebay or equivalent upstream pretreatment must be provided
- Minimum length to width ratio for the pond is 1.5:1
- Maximum depth of the permanent pool should not exceed 8 feet
- Vegetated side slopes to the pond should not exceed 3:1 (h:v)

ADVANTAGES / BENEFITS:

- Moderate to high removal rate of urban pollutants
- High community acceptance
- Opportunity for wildlife habitat

DISADVANTAGES / LIMITATIONS:

- Potential for thermal impacts/downstream warming
- Dam height restrictions for high relief areas
- Pond drainage can be problematic for low relief terrain

MAINTENANCE REQUIREMENTS:

- Remove debris from inlet and outlet structures
- Maintain side slopes / remove invasive vegetation
- Monitor sediment accumulation and remove periodically
- Dam inspection and maintenance

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/30%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
70%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L-M	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *10-25 acres min.*

Soils: *Hydrologic group 'A' and 'B' soils may require pond liner*

Other Considerations:

- *Outlet Clogging*
- *Safety Bench*
- *Landscaping*
- *Hotspot areas*

L=Low **M**=Moderate **H**=High

5.2.21.1 General Description

Storm water ponds (also referred to as *retention ponds*, *wet ponds*, *or wet extended detention ponds*) are constructed storm water retention basins that have a permanent (dead storage) pool of water throughout the year. They can be created by excavating an already existing natural depression or through the construction of embankments.

In a storm water pond, runoff from each rain event is detained and treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from resuspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. The upper stages of a storm water pond are designed to provide extended detention of the 1-year storm for downstream streambank protection, as well as normal detention of larger storm events to meet Q_f requirements.

Storm water ponds are among the most cost-effective and widely used storm water practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned and located properly.

There are several different variants of storm water pond design, the most common of which include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple storm water ponds can be placed in series or parallel to increase performance or meet site design constraints. Below are descriptions of each design variant:

- **Wet Pond** – Wet ponds are storm water basins constructed with a permanent (dead storage) pool of water equal to the water quality volume. Storm water runoff displaces the water already present in the pool. Temporary storage (live storage) can be provided above the permanent pool elevation for larger flows.
- **Wet Extended Detention (ED) Pond** – A wet extended detention pond is a wet pond where the water quality volume is split evenly between the permanent pool and extended detention (ED) storage provided above the permanent pool. During storm events, water is detained above the permanent pool and released over 24 hours. This design has similar pollutant removal to a traditional wet pond, but consumes less space.
- **Micropool Extended Detention (ED) Pond** – The micropool extended detention pond is a variation of the extended detention wet pond where only a small "micropool" is maintained at the outlet to the pond. The outlet structure is sized to detain the water quality volume for 24 hours. The micropool prevents resuspension of previously settled sediments and also prevents clogging of the low flow orifice.
- **Multiple Pond Systems** – Multiple pond systems consist of constructed facilities that provide water quality and quantity volume storage in two or more cells. The additional cells can create longer pollutant removal pathways and improved downstream protection.

Figure 5.2.21-1 shows a number of examples of storm water pond variants. Section 5.2.21.8 provides plan view and profile schematics for the design of a wet pond, wet extended detention pond, micropool extended detention pond, and multiple pond system.

Conventional dry detention basins do not provide a permanent pool and are **not recommended** for general application use to meet water quality criteria, as they fail to demonstrate an ability to meet the majority of the water quality goals. In addition, dry detention basins are prone to clogging and resuspension of previously settled solids and require a higher frequency of maintenance than wet ponds if used for untreated storm water flows. These facilities can be used in combination with appropriate water quality controls to provide streambank protection, and overbank and extreme flood storage. Please see a further discussion in subsection 5.2.9 (*Dry Detention Basins*).

Figure 5.2.21-1 Storm Water Pond Examples

5.2.21.2 Storm Water Management Suitability

Storm water ponds are designed to control both storm water quantity and quality. Thus, a storm water pond can be used to address all of the *integrated storm water sizing criteria* for a given drainage area.

Water Quality

Ponds treat incoming storm water runoff by physical, biological, and chemical processes. The primary removal mechanism is gravitational settling of particulates, organic matter, metals, bacteria, and organics as storm water runoff resides in the pond. Another mechanism for pollutant removal is uptake by algae and wetland plants in the permanent pool – particularly of nutrients. Volatilization and chemical activity also work to break down and eliminate a number of other storm water contaminants such as hydrocarbons.

Section 5.2.21.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

A portion of the storage volume above the permanent pool in a storm water pond can be used to provide control of the streambank protection volume (SP_v). This is accomplished by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention).

On-Site Flood Control

A storm water pond can also provide detention storage above the permanent pool to reduce the post-development peak flow to pre-development levels, if required.

Downstream Flood Control

In situations where it is required, storm water ponds can also be used to provide detention to control the 100-year storm peak flow downstream. Where this is not required, the pond structure is designed to safely pass extreme storm flows.

5.2.21.3 Pollutant Removal Capabilities

All of the storm water pond design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed ponds can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 50%**
- **Total Nitrogen – 30%**
- **Fecal Coliform – 70% (if no resident waterfowl population present)**
- **Heavy Metals – 50%**

For additional information and data on pollutant removal capabilities for storm water ponds, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Storm Water Best Management Practices (BMP) Database at www.bmpdatabase.org

5.2.21.4 Application and Site Feasibility Criteria

Storm water ponds are generally applicable to most types of new development and redevelopment, and can be used in both residential and nonresidential areas. Ponds can also be used in retrofit situations. The following criteria should be evaluated to ensure the suitability of a storm water pond for meeting storm water management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra-Urban Areas – Land requirements may preclude use
- Regional Storm Water Control – YES
- Hotspot Runoff - YES

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – A minimum of 25 acres is needed for wet pond and extended detention wet pond to maintain a permanent pool, 10 acres minimum for extended detention micropool pond. A smaller drainage area may be acceptable with an adequate water balance and anti-clogging device.
- Space Required – Approximately 2 to 3% of the tributary drainage area

- Site Slope – There should not be more than 15% slope across the pond site.
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 6 to 8 feet
- Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer or when treating a hotspot, a separation distance of 2 feet is required between the bottom of the pond and the elevation of the seasonally high water table.
- Soils – Underlying soils of hydrologic group "C" or "D" should be adequate to maintain a permanent pool. Most group "A" soils and some group "B" soils will require a pond liner. *Evaluation of soils should be based upon an actual subsurface analysis and permeability tests*.

Other Constraints / Considerations

- Local Aquatic Habitat – Consideration should be given to the thermal influence of storm water pond outflows on downstream local aquatic habitats.

5.2.21.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a storm water pond facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- Storm water ponds should have a minimum contributing drainage area of 25 acres or more for wet pond or extended detention wet pond to maintain a permanent pool. For an extended detention micropool pond, the minimum drainage area is 10 acres. A smaller drainage area can be considered when water availability can be confirmed (such as from a groundwater source or areas with a high water table). In these cases a water balance may be performed (see subsection 2.1.11 for details). Ensure that an appropriate anti-clogging device is provided for the pond outlet.
- A storm water pond should be sited such that the topography allows for maximum runoff storage at minimum excavation or construction costs. Pond siting should also take into account the location and use of other site features such as buffers and undisturbed natural areas and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.
- Storm water ponds should not be located on steep (>15%) or unstable slopes.
- Storm water ponds cannot be located within a stream or any other navigable waters of the U.S., including wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit.
- Minimum setback requirements for storm water pond facilities measured from the easement line that defines the pond site (when not specified by local ordinance or criteria):
  - From a property line – 10 feet
  - From a private well – 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
  - From a septic system tank/leach field/spray area – 50 feet
- All utilities should be located outside of the pond/basin site.

B. GENERAL DESIGN

- A well-designed storm water pond consists of:
  1. Permanent pool of water,
  2. Overlying zone in which runoff control volumes are stored, and
  3. Shallow littoral zone (aquatic bench) along the edge of the permanent pool that acts as a biological filter.

- In addition, all storm water pond designs need to include a sediment forebay at all major inflows to the basin to allow heavier sediments to drop out of suspension before the runoff enters the permanent pool. (A sediment forebay schematic can be found in Appendix C)
- Additional pond design features include an emergency spillway, maintenance access, safety bench, pond buffer, and appropriate native landscaping.

Figures 5.2.21-4 thru 5.2.21-7 in subsection 5.2.21.8 provide plan view and profile schematics for the design of a wet pond, extended detention wet pond, extended detention micropool pond and multiple pond system.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

In general, pond designs are unique for each site and application. However, there are number of geometric ratios and limiting depths for pond design that must be observed for adequate pollutant removal, ease of maintenance, and improved safety.

- Permanent pool volume is typically sized as follows:
  - Standard wet ponds: 100% of the water quality treatment volume (1.0 WQ_v)
  - Extended detention wet ponds: 50% of the water quality treatment volume (0.5 WQ_v)
  - extended detention micropool ponds: Approximately 0.1 inch per impervious acre
- Proper geometric design is essential to prevent hydraulic short-circuiting (unequal distribution of inflow), which results in the failure of the pond to achieve adequate levels of pollutant removal. The minimum length-to-width ratio for the permanent pool shape is 1.5:1, and should ideally be greater than 3:1 to avoid short-circuiting. In addition, ponds should be wedge-shaped when possible so that flow enters the pond and gradually spreads out, improving the sedimentation process. Baffles, pond shaping or islands can be added within the permanent pool to increase the flow path.
- Maximum depth of the permanent pool should generally not exceed 8 feet to avoid stratification and anoxic conditions. Minimum depth for the pond bottom should be 3 to 4 feet. Deeper depths near the outlet will yield cooler bottom water discharges that may mitigate downstream thermal effects.
- Side slopes to the pond should not usually exceed 3:1 (h:v) without safety precautions or if mowing is anticipated and should terminate on a safety bench (see Figure 5.2.21-2). The safety bench requirement may be waived if slopes are 4:1 or gentler. All side slopes should be verified with a geotechnical evaluation to ensure slope stability.
- The perimeter of all deep pool areas (4 feet or greater in depth) should be surrounded by two benches: safety and aquatic. For larger ponds, a safety bench extends approximately 15 feet outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench should be 6%. An aquatic bench extends inward from the normal pool edge (15 feet on average) and has a maximum depth of 18 inches below the normal pool water surface elevation (see Figure 5.2.21-2).

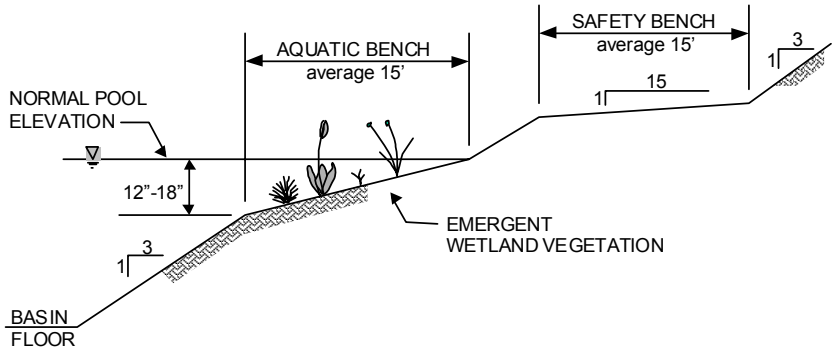


Figure 5.2.21-2 Typical Storm Water Pond Geometry Criteria

- The contours and shape of the permanent pool should be irregular to provide a more natural landscaping effect.

D. PRETREATMENT / INLETS

- Each pond should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the storm water flow prior to dispersal in a larger permanent pool. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. In some design configurations, the pretreatment volume may be located within the permanent pool.
- The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The pretreatment storage volume is part of the total WQ_v requirement and may be subtracted from WQ_v for permanent pool sizing.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Inflow pipe, channel velocities, and exit velocities from the forebay must be nonerosive.

E. OUTLET STRUCTURES

- Flow control from a storm water pond is typically accomplished with the use of a concrete or corrugated aluminum, aluminized steel, or HDPE riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the bottom of the pond with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 5.2.21-3). The riser should be located within the embankment for maintenance access, safety and aesthetics.

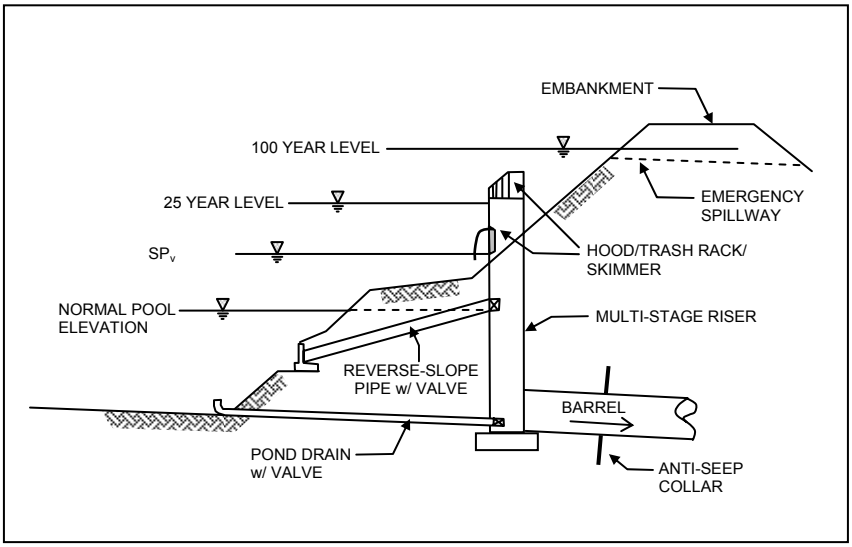


Figure 5.2.21-3 Typical Pond Outlet Structure

- A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, streambank protection, and on-site flood control runoff volumes. The number of orifices can vary and is usually a function of the pond design.
- Embankments 6 feet in height or greater shall be designed per Texas Commission on Environmental Quality guidelines for Dam Safety. See Appendix H.

- For example, a wet pond riser configuration is typically comprised of a streambank protection outlet (usually an orifice) and on-site flood control outlet (often a slot or weir). The streambank protection orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the streambank protection orifice. Thus an off-line wet pond providing *only* water quality treatment can use a simple overflow weir as the outlet structure.
- In the case of an extended detention wet pond or extended detention micropool pond, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality extended detention volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the streambank protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the streambank protection storage volume over a 24-hour period.
- Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.
- The water quality outlet (if design is for an extended detention wet or extended detention micropool pond) and streambank protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.
- Higher flows (On-Site and Downstream Flood Control) pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools, or pads, or other energy dissipators are to be placed at the outlet of the barrel to prevent scouring and erosion. If a pond daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Section 4.7 (*Energy Dissipation Design*) for more guidance.
- Each pond must have a bottom drain pipe with an adjustable valve that can completely or partially drain the pond within 24 hours.
- The pond drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

See the design procedures in 5.2.21.6 as well as Section 2.2 (*Storage Design*) and Section 4.6 (*Outlet Structures*) for additional information and specifications on pond routing and outlet works.

F. EMERGENCY SPILLWAY

- An emergency spillway is to be included in the storm water pond design to safely pass the extreme flood flow. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges. All local and state dam safety requirements should be met.

- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

G. MAINTENANCE ACCESS

- A maintenance right of way or easement must be provided to a pond from a public road or easement. Maintenance access should be at least 12 feet wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser is to be provided by lockable manhole covers, and manhole steps should be within easy reach of valves and other controls.

H. SAFETY FEATURES

- All embankments and spillways must be designed to State of Texas guidelines for dam safety (see Appendix H).
- Fencing of ponds is not generally desirable, but may be required by the local review authority. A preferred method is to manage the contours of the pond through the inclusion of a safety bench (see above) to eliminate dropoffs and reduce the potential for accidental drowning. In addition, the safety bench may be landscaped to deter access to the pool.
- The principal spillway opening should not permit access by small children, and endwalls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent access. Warning signs should be posted near the pond to prohibit swimming and fishing in the facility.

I. LANDSCAPING

- Aquatic vegetation can play an important role in pollutant removal in a storm water pond. In addition, vegetation can enhance the appearance of the pond, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris. Therefore, wetland plants should be encouraged in a pond design, along the aquatic bench (fringe wetlands), the safety bench and side slopes (ED ponds), and within shallow areas of the pool itself. The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within 6 inches (plus or minus) of the normal pool elevation. Additional information on establishing wetland vegetation and appropriate wetland species for North Central Texas can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.
- Woody vegetation may not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- A pond buffer should be provided that extends 25 feet outward from the maximum water surface elevation of the pond. The pond buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall storm water management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs and native ground covers.
- The soils of a pond buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.

- Fish such as Gambusia affinis can be stocked in a pond to aid in mosquito prevention.
- A fountain or solar-powered aerator may be used for oxygenation of water in the permanent pool.
- Compatible multi-objective use of storm water pond locations is strongly encouraged.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Maximum normal pool depth is limited; providing pond drain can be problematic
- High Relief – Embankment heights restricted
- Karst – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required

Soils

- Hydrologic group "A" soils generally require pond liner; group "B" soils may require infiltration testing

Special Downstream Watershed Considerations

- Local Aquatic Habitat – extended detention micropool pond best alternative; design wet ponds and extended detention wet ponds offline and provide shading to minimize thermal impact; limit WQ_v-ED to 12 hours
- Aquifer Protection – Reduce potential groundwater contamination by preventing infiltration of hotspot runoff. May require liner for type "A" and "B" soils; pretreat hotspots; 2 to 4 foot separation distance from water table
- Swimming Area/Shellfish – Design for geese prevention (see Appendix F); provide 48-hour extended detention for maximum coliform dieoff.

Dams

Dam construction for storm water ponds can take a variety of forms. Large dams that are over six feet in height are regulated by the State of Texas (See Appendix H). Small dams are not as tightly regulated, but require careful attention to design and construction details to ensure that they function properly throughout their designed economic life.

The most commonly used material for small dam construction is earth fill, but structural concrete can also be used. For on-site storm water controls in high density areas of development or where land values are very costly, the use of a structural concrete dam can save significant amounts of land while making a much more aesthetically appealing outfall structure that the typical riser and barrel assembly.

- **General**
  - The dam area shall be cleared, grubbed and stripped of all vegetative material and topsoil prior to dam construction.
- **Earth Dams**
  - The dam construction plans shall indicate allowable soil materials to be used, compaction required, locations of core trenches if used, any sub-drainage facilities to be installed to control seepage, plus horizontal and vertical dimensions of the earthen structure.
  - The sub-grade of the dam shall be scarified prior to the placement and compaction of the first lift of soil backfill to ensure a good bond between the existing soil and the earthen dam.
  - Placement of earth fill shall be in controlled lifts with proper compaction.
  - Placement of spillway or outflow pipes through the dam shall be per the plan details, with proper backfill and compaction of any excavated trenches. Hydraulic flooding or other compaction methods of saturated soil shall not be allowed.

- Topsoil and soil additives necessary for the establishment of permanent ground cover above the normal water surface elevation and on the downstream side of the dam shall be installed and seeded as soon as practical to avoid rilling and erosion of the dam's earthen embankment.
- Do not plant trees or large shrubs on the earth dam, as their root systems cause seepage and damage to the structure.

- **Concrete Dams**
  - Concrete dams shall be designed and built in accordance with the American Concrete Institute's (ACI) latest guidelines for Environmental Engineering Concrete Structures. Particular attention shall be paid to water tightness, crack control, concrete materials and construction practices.
  - The construction plans shall indicate materials, plus horizontal and vertical dimensions necessary for the construction of the dam. Details and information shall be provided on joint types and spacing to be used.
  - At least one-half of the water surface perimeter of the pond at normal pool elevation shall be constructed with a vegetated earthen embankment.
  - Principal and emergency spillways can be incorporated into a weir overflow over the dam if splash pads or another type of control structure is provided to protect the downstream toe of the concrete structure.
  - Placement of drain valves, overflow controls and other penetrations of the concrete wall shall not be located on the same vertical line to prevent creating a weakened plane where uncontrolled cracks can form. Locations should also anticipate operation during storm events when overflow weirs will be operating.

5.2.21.6 Design Procedures

Step 1. Compute runoff control volumes from the *integrated* Design Approach

Calculate the Water Quality Volume (WQ_v), Streambank Protection Volume (SP_v), and the Flood Protection Storm (Q_f). Design volume should be increased by 15% for extended detention ponds.

Details on the *integrated* Design Approach are found in Section 1.2.

Step 2. Determine if the development site and conditions are appropriate for the use of a storm water pond

Consider the Application and Site Feasibility Criteria in subsections 5.2.21.4 and 5.2.21.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 5.2.21.5-J. (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The forebay storage volume counts toward the total WQ_v requirement and may be subtracted from the WQ_v for subsequent calculations.

Step 5. Determine permanent pool volume (and water quality extended detention volume)

Wet Pond: Size permanent pool volume to 1.0 WQ_v

Extended Detention Wet Pond: Size permanent pool volume to 0.5 WQ_v. Size extended detention volume to 0.5 WQ_v.

Extended Detention Micropool Pond: Size permanent pool volume to 25 to 30% of WQ_v. Size extended detention volume to remainder of WQ_v.

Step 6. Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for permanent pool (and water quality extended detention if extended detention wet pond or extended detention micropool pond)

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond.

- Include safety and aquatic benches.
- Set WQ_v permanent pool elevation (and WQ_v-ED elevation for extended detention wet and extended detention micropool pond) based on volumes calculated earlier.

See subsection 5.2.21.5-C (Physical Specifications / Geometry) for more details.

Step 7. Compute extended detention orifice release rate(s) and size(s), and establish SP_v elevation

Wet Pond: The SP_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

Extended Detention Wet Pond and Extended Detention Micropool Pond: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. Adjustable gate valves can also be used to achieve this equivalent diameter. The SP_v elevation is then determined from the stage-storage relationship. The invert of the streambank protection orifice is located at the water quality extended detention elevation, and the orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

Step 8. Calculate Q_p release rate and water surface elevation

Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the deisgn storm.

Step 9. Design embankment(s) and spillway(s)

Size emergency spillway, calculate 100-year water surface elevation, set top of embankment elevation, and analyze safe passage of the 100-year flood.

At final design, provide safe passage for the 100-year event.

Step 10. Investigate potential pond hazard classification

The design and construction of storm water management ponds are required to follow the latest version of the State of Texas Administrative Code for dams and reservoirs (see Appendix H).

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See subsection 5.2.21.5-D through H for more details.

Step 12. Prepare Vegetation and Landscaping Plan

A landscaping plan for a storm water pond and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See subsection 5.2.21.5-I (Landscaping) and Appendix F for more details.

See Appendix D-1 for a Storm Water Pond Design Example

5.2.21.7 Inspection and Maintenance Requirements

Table 5.2.21-1 Typical Maintenance Activities for Ponds
(Source: WMI, 1997)

Activity	Schedule
• Clean and remove debris from inlet and outlet structures. • Mow side slopes. • Check visually for illegal dumping or other pollutants.	Monthly
• If wetland components are included, inspect for invasive vegetation.	Semiannual Inspection
• Inspect for damage, paying particular attention to the control structure. • Check for signs of eutrophic conditions. • Note signs of hydrocarbon build-up, and remove appropriately. • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational. • Check all control gates, valves or other mechanical devices. • Check downstream face of dam for seepage (earth and concrete), settling (earth) and cracking (concrete).	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Perform wetland plant management and harvesting.	Annually (if needed)
• Remove sediment from the forebay.	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly, or the pond becomes eutrophic.	10 to 20 years or after 25% of the permanent pool volume has been lost

Additional Maintenance Considerations and Requirements

- A sediment marker should be located in the forebay to determine when sediment removal is required.
- Sediments excavated from storm water ponds that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present.
- Periodic mowing of the pond buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.
- Care should be exercised during pond drawdowns to prevent downstream discharge of sediments, anoxic water, or high flows with erosive velocities. The approving jurisdiction should be notified before draining a storm water pond.

Regular inspection and maintenance is critical to the effective operation of storm water ponds as designed. Maintenance responsibility for a pond and its buffer should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

5.2.21.8 Example Schematics

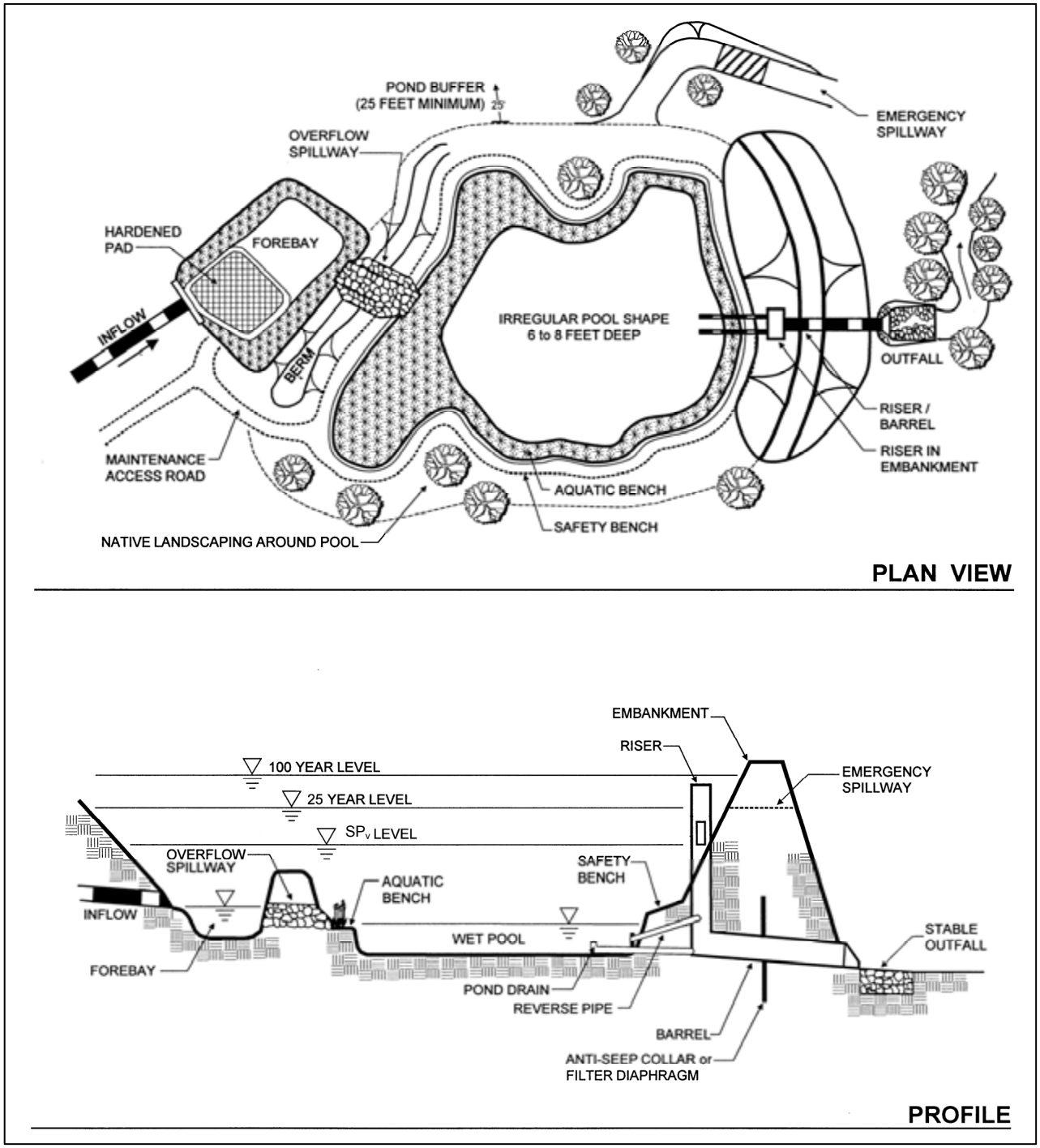


Figure 5.2.21-4 Schematic of Wet Pond
(Source: Center for Watershed Protection)

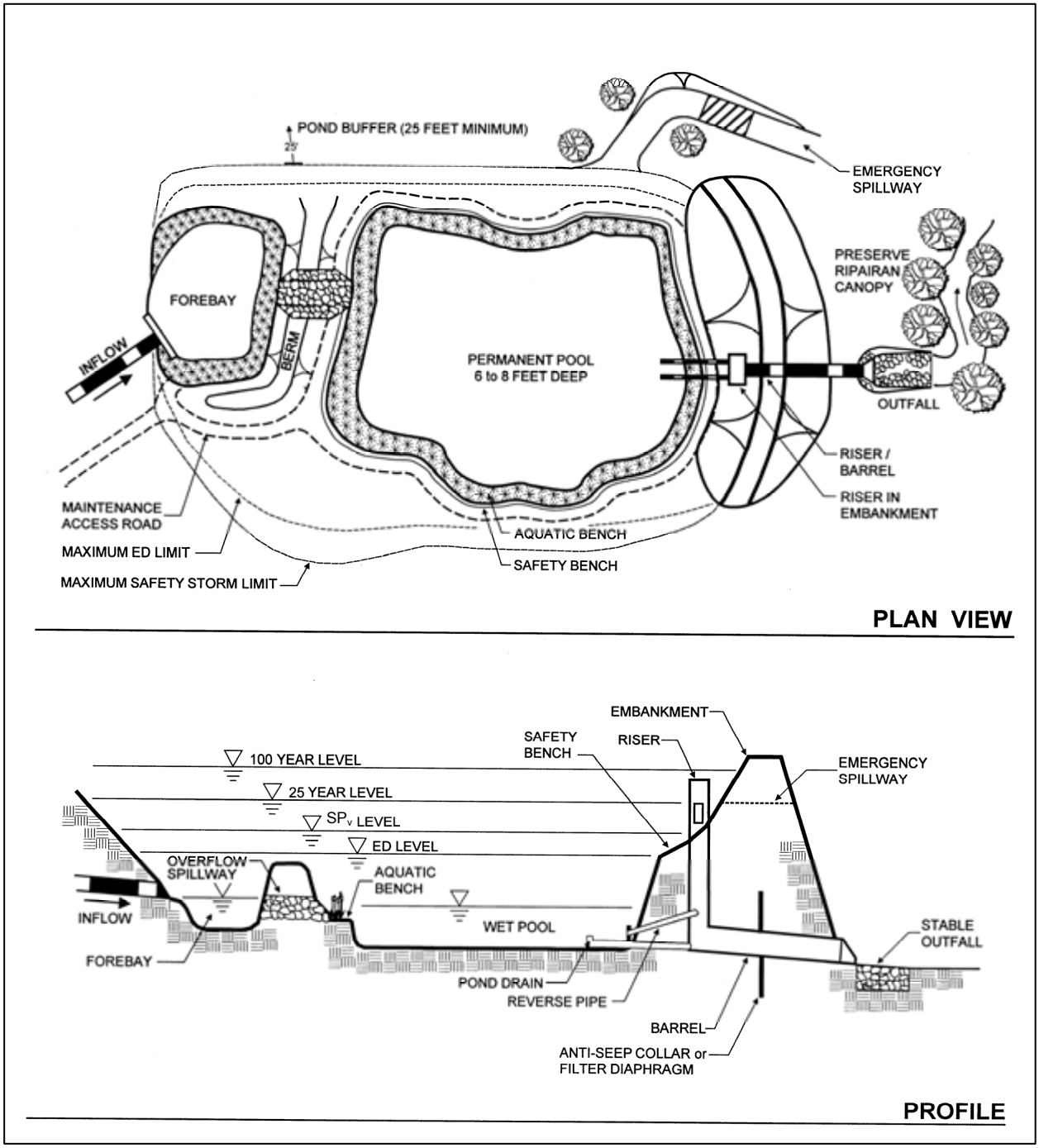


Figure 5.2.21-5 Schematic of Wet Extended Detention Pond
(Source: Center for Watershed Protection)

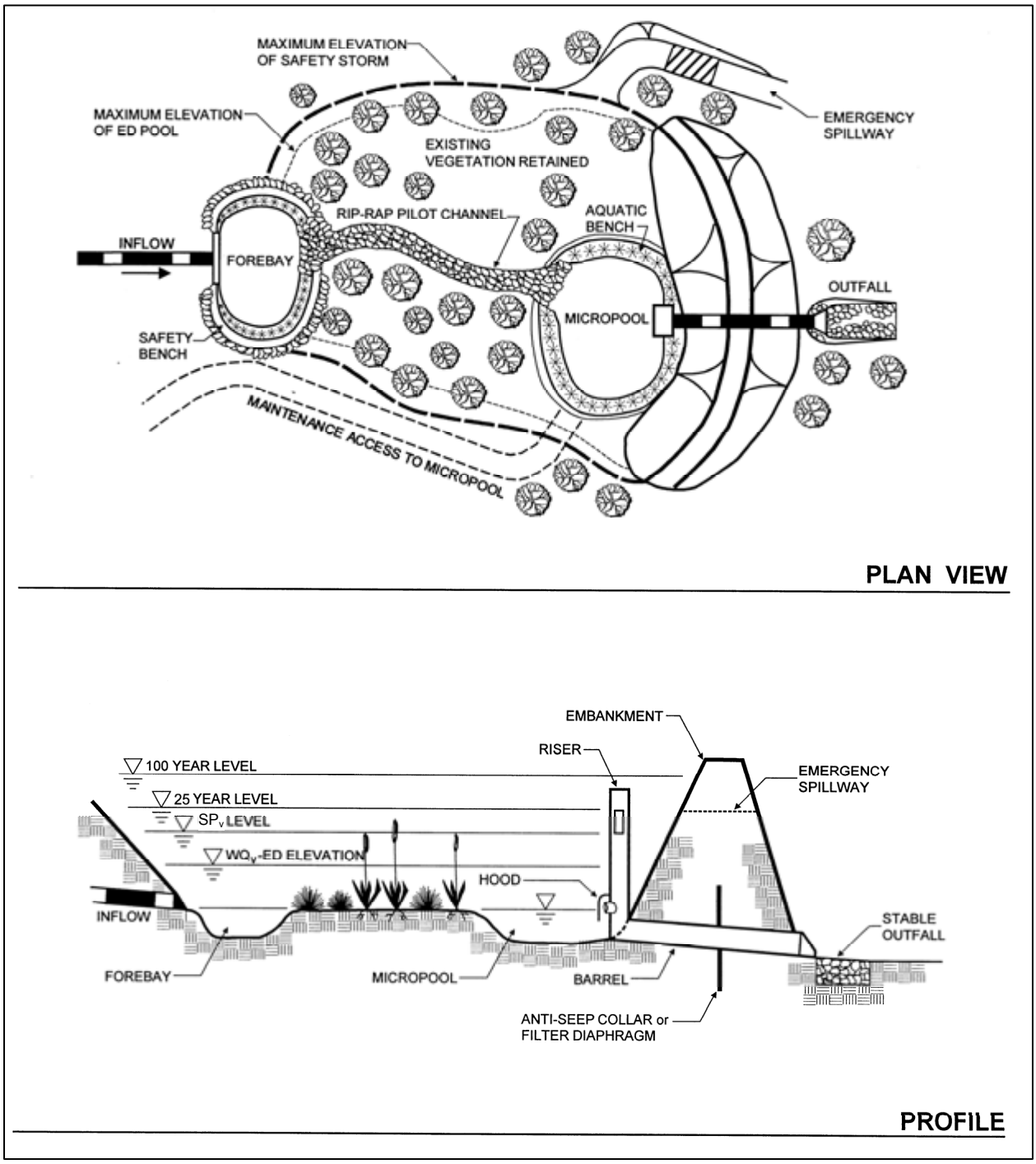


Figure 5.2.21-6 Schematic of Micropool Extended Detention Pond
(Source: Center for Watershed Protection)

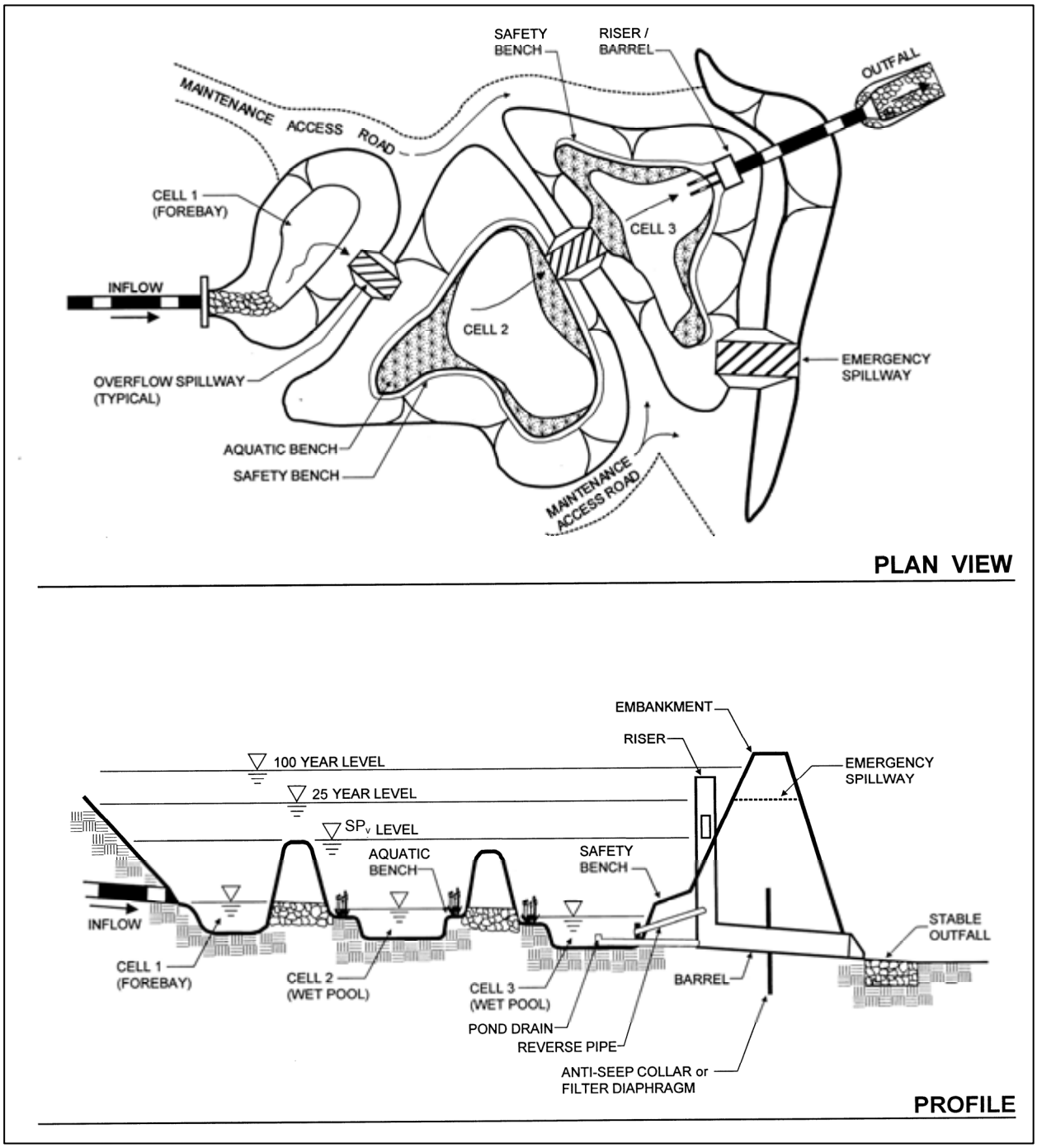


Figure 5.2.21-7 Schematic of Multiple Pond System
(Source: Center for Watershed Protection)

5.2.21.9 Design Forms

Design Procedure Form: Storm Water Ponds

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v Volume requirements
Compute Runoff Coefficient, R_v — R_v = _____
Compute WQ_v Volume requirements — WQ_v = _____ acre-ft

1b. Compute SP_v — SP_v = _____ acre-ft
Compute average release rate — release rate = _____ cfs
Compute Q_p (Required 100-year detention volume) — Q_p = _____ acre-ft
Add 15% to the required Q_p volume (if ED) — Q_p* 15% = _____ acre-ft
Compute (as necessary) Q_f — Q_f = _____ cfs

STORM WATER POND DESIGN

2. Is the use of storm water pond appropriate? — **See subsection 5.2.21.4 and 5.2.21.5-A**

3. Confirm local design criteria and applicability

4. Pretretament volume
Vol_{pre} = I(0.1")(1'/12") — Vol_{pre} = _____ acre-ft

5. Allocation of Permanent Pool Volume and ED Volume

Wet Pond: $Vol_{pool} = WQ_v$ — Vol_{pool} = _____ acre-ft

Wet ED Pond: $Vol_{pool} = 0.5(WQ_v)$ — Vol_{pool} = _____ acre-ft
$Vol_{ED} = 0.5(WQ_v)$ — Vol_{ED} = _____ acre-ft

Micropool ED Pond: $Vol_{pool} = 0.25(WQ_v)$ — Vol_{pool} = _____ acre-ft
$Vol_{ED} = 0.75(WQ_v)$ — Vol_{ED} = _____ acre-ft

6. Conduct grading and determine storage available for permanent pool (and WQ_v-ED volume if applicable) — Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume acre-ft	Volume above Permanent Pool acre-ft

7. WQ_v Orifice Computations
   Average ED release rate (if applicable) — release rate = _____ cfs
   Average head, h = (ED elev. - Permanent pool elev.) / 2 — h = _____ ft
   Area of orifice from orifice equation — A = _____ ft^2
   $Q = CA(2gh)^{0.5}$ — diameter = _____ in
   factor = _____ $(h)^{0.5}$

   Discharge equation $Q = (h)^{0.5}$

   Compute release rate for SP_v-ED control and establish SP_v elevation — WSEL = _____ ft-NGVD
   Release rate = — release rate = _____ cfs
   Aerage head, h = SP_v elev. - Permanent pool elev.) / 2 — h = _____ ft-NGVD
   Area of orifice from orifice equatin — A = _____ ft^2
   $Q = CA(2gh)^{0.5}$ — diameter = _____ in
   Discharge equation $Q = (h)^{0.5}$ — factor = _____ $(h)^{0.5}$

8. Calculate Qp release rate and WSEL — **Set up a stage-storage-discharge relationship**

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	SPv-ED	High Storage		Inlet	Pipe	Spillway	Storage
				Orif.	Weir				
MSL	ac-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	HQ	HQ	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	Q(cfs)

   Q_p = pre-dev. Peak discharge - (WQ_v-ED release + SP_v-ED release) — Q_p = _____ cfs

   Maximum head = — H = _____ ft
   Use weir equation for slot length ($Q = CLH^{3/2}$) — L = _____ ft

   Check inlet condition — **Use culvert charts**
   Check outlet condition — **(section 4.3)**

9. Size emergency spillway, calculate 100-year WSEL and set top of embankment elevation
   $WSEL_{25}$ = _____ ft
   $WSEL_{100}$ = _____ ft
   Q_{ES} = _____ cfs
   Q_{PS} = _____ cfs

10. Investigate potential pond hazard classification — **See Appendix H**

11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features. — **See subsection 3.2.15 - D through H**

12. Attach landscaping plan — **See Appendix F**

5.2.22 Green Roof

Structural Storm Water Control

Description: A green roof uses a small amount of substrate over an impermeable membrane to support a covering of plants. The green roof slows down runoff from the otherwise impervious roof surface as well as moderating rooftop temperatures. With the right plants, a green roof will also provide aesthetic or habitat benefits. Green roofs have been used in Europe for decades.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Relatively new in North America
- Potential for high failure rate if poorly designed, poorly constructed, not adequately maintained Minimum length to width ratio for the pond is 1.5:1

ADVANTAGES / BENEFITS:

- Provides reduction in runoff volume
- Higher initial cost when compared to conventional roofs, but potential for lower life cycle costs through longevity

DISADVANTAGES / LIMITATIONS:

- Requires additional roof support
- Requires more maintenance than regular roofs
- Special attention to design and construction needed
- Requires close coordination with plant specialists
- Potential for leakage due to plant roots penetrating membrane.

POLLUTANT REMOVAL

85%	**Total Suspended Solids**
95%/16%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
25%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [P] **Water Quality Protection**
- [S] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [L] **Land Requirement**
- [H] **Capital Cost**
- [H] **Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *No restrictions.*

Soils: *No restrictions*

Other Considerations:

- *Hotspot Areas*

L=Low **M**=Moderate **H**=High

5.2.22.1 General Description

Green roofs (also referred to as *ecoroofs*, *roof gardens, or roof meadows*) are vegetated roofs used in place of conventional roofing, such as gravel-ballasted roofs. They are used as part of sustainable development initiatives, along with narrow streets, permeable pavement, and various infiltration devices. There are two main types of green roofs. The first is what is called roof gardens or intensive green roofs. They may be thought of as a garden on the roof. They have a greater diversity of plants, including trees and shrubs, but require deeper soil, increased load bearing capacity, and require more maintenance. The second has been referred to as roof meadows or extensive green roofs. The vegetation is limited and similar to an alpine meadow, requiring less soil depth and minimal maintenance. Due to the considerably greater costs and structural design requirements, only the second type of green roof, the roof meadow or extensive type is discussed in this manual.

The extensive green roof is designed to control low-intensity storms by intercepting and retaining or storing water until the peak storm event has passed. The plants intercept and delay runoff by capturing and holding precipitation in the foliage, absorbing water in the root zone, and slowing the velocity of direct runoff by increasing retardance to flow and extending the flowpath through the vegetation. Water is also stored and evaporated from the growing media. Green roofs can capture and evaporate up to 100 percent of the incident precipitation, depending on the roof design and the storm characteristics.

Monitoring in Pennsylvania, for instance, showed reductions of approximately 2/3 in runoff from a green roof (15.5 inches runoff from 44 inches of rainfall). Furthermore, runoff was negligible for storm events of less than 0.6 inches. A study done for Portland, Oregon, indicated a reduction in storm water discharges from the downtown area of between 11 and 15% annually if half of the roofs in the downtown area were retrofitted as green roofs.

Green roofs also:

- reduce the temperature of runoff,
- reduce the “heat island” effect of urban buildings,
- help insulate the building,
- improve visual aesthetics,
- protect roofs from weather,
- improve building insulation,
- reduce noise,
- and provide habitat for wildlife.

As with a conventional roof, a green roof must safely drain runoff from the roof. It may be desirable to drain the runoff to a rainwater harvesting system such as (rainbarrels or cisterns), or other storm water facilities such as planters and swales.

Significant removals of heavy metals by green roofs have been reported, but there is not enough evidence to include removal rates at this time.

5.2.22.2 Design Criteria and Specifications

- For either new installations or retrofits, an architect or structural engineer must be consulted to determine whether the building can provide the structural support needed for a green roof.
- Generally, the building structure must be adequate to hold an additional 10 to 25 pounds per square foot (psf) saturated weight, depending on the vegetation and growth medium that will be used. (This is in addition to snow load requirements.) An existing rock ballast roof may be structurally sufficient to hold a 10-12 psf green roof, since ballast typically weighs 10-12 psf.

- Green roofs can be used on flat or pitched roofs up to 40 percent. Although, on a roof slope greater than 20 degrees, the green roof installer needs to ensure that the plant layer does not slip or slump through its own weight, especially when it becomes wet. Horizontal strapping, wood, plastic, or metal, may be necessary. Some commercial support grid systems are also available for this purpose.
- A green roof typically consists of several layers, as shown in Figure 5.2.22-1. A waterproof membrane is placed over the roof's structure. A root barrier is placed on top of the membrane to prevent roots from penetrating the membrane and causing leaks. A layer for drainage is installed above this, followed by the growth media. The vegetation is then planted to form the top layer. Details of the various layers are given below.
- Waterproof membranes are made of various materials, such as synthetic rubber (EPDM), hypolan (CPSE), reinforced PVC, or modified asphalts (bitumens). The membranes are available in various forms, liquid, sheets, or rolls. Check with the manufacturer to determine their strength and functional characteristics of the membrane under consideration.
- Root barriers are made of dense materials or are treated with copper or other materials that inhibit root penetration, protecting the waterproof membrane from being breached. A root barrier may not be necessary for synthetic rubber or reinforced PVC membranes, but will likely be needed for asphalt mixtures. Check with the manufacturer to determine if a root barrier is required for a particular product.
- The drainage layer of a green roof is usually constructed of various forms of plastic sheeting, a layer of gravel, or in some cases, the growth medium.
- The growth medium is generally 2 to 6 inches thick and made of a material that drains relatively quickly. Commercial mixtures containing coir (coconut fiber), pumice, or expanded clay are available. Sand, gravel, crushed brick, and peat are also commonly used. Suppliers recommend limiting organic material to less than 33% to reduce fire hazards. The City of Portland, Oregon has found a mix of 1/3 topsoil, 1/3 compost, and 1/3 perlite may be sufficient for many applications. Growth media can weigh from 16 to 35 psf when saturated depending on the type (intensive/extensive), with the most typical range being from 10-25 psf.
- When dry, all of the growth media are light-weight and prone to wind erosion. It is important to keep media covered before planting and ensure good coverage after vegetation is established.
- Selecting the right vegetation is critical to minimize maintenance requirements. Due to the shallowness of the growing medium and the extreme desert-like microclimate on many roofs, plants are typically alpine, dryland, or indigenous. Ideally, the vegetation should be:
  - Drought-tolerant, requiring little or no irrigation after establishment
  - Self-sustaining, without fertilizers, pesticides, or herbicides
  - Able to withstand heat, cold, and high winds
  - Shallow root structure
  - Low growing, needing little or no mowing or trimming
  - Fire resistant
  - Perennial or self propagating, able to spread and cover blank spots by itself

  Visit www.txsmartscape.com to look up plants meeting the above criteria.
- A mix of sedum/succulent plant communities is recommended because they possess many of these attributes. Certain wildflowers, herbs, forbs, grasses, mosses, and other low groundcovers can also be used to provide additional habitat benefits or aesthetics; however, these plants need more watering and maintenance to survive and keep their appearance.
- Green roof vegetation is usually established by one or more of the following methods: seeding, cuttings, vegetation mats, and plugs/potted plants.
  - Seeds can be either hand sown or broadcast in a slurry (hydraseeded). Seeding takes longer to establish and requires more weeding, erosion control, and watering than the other methods.

- Cuttings or sprigs are small plant sections. They are hand sown and require more weeding, erosion control, and watering than mats.
- Vegetation mats are sod-like mats that achieve full plant coverage very quickly. They provide immediate erosion control, do not need mulch, and minimize weed intrusion. They generally require less ongoing maintenance than the other methods.
- Plugs or potted plants may provide more design flexibility than mats. However, they take longer to achieve full coverage, are more prone to erosion, need more watering during establishment, require mulching, and more weeding.

- Green roof vegetation is most easily established during the spring or fall.
- Irrigation is necessary during the establishment period and possibly during drought conditions, regardless of the planting method used. The goal is to minimize the need for irrigation by paying close attention to plant selection, soil, and various roof characteristics.
- Installation costs for green roofs generally run from $10 to $25 per square foot, as compared to $3 to $20 per square foot for a conventional roof. However, the longer lifespan of a green roof (reportedly 40 years or up to twice as long as a conventional roof) and lower maintenance costs offset this.
- Provide controlled overflow point(s) to prevent overloading of roof.

5.2.22.3 Inspection and Maintenance Requirements

Table 5.2.22-1 Typical Maintenance Activities for Green Roofs

Activity	Schedule
• Watering to help establish vegetation	As needed
• Replant to cover bare spots or dead plants	Monthly
• Weeding (as needed, based on inspection)	Two or three times yearly
• Water and mowing to prevent fire hazards (if grasses or similar plants are used)	As needed
• Inspect drains for clogging	Twice per year
• Inspect the roof for leakage	Annually, or as needed
• If leaks occur, remove and stockpile vegetation, growth media, and drainage layer. Replace membrane and root barrier, followed by stockpiled material.	Upon failure

5.2.22.4 Example Schematic

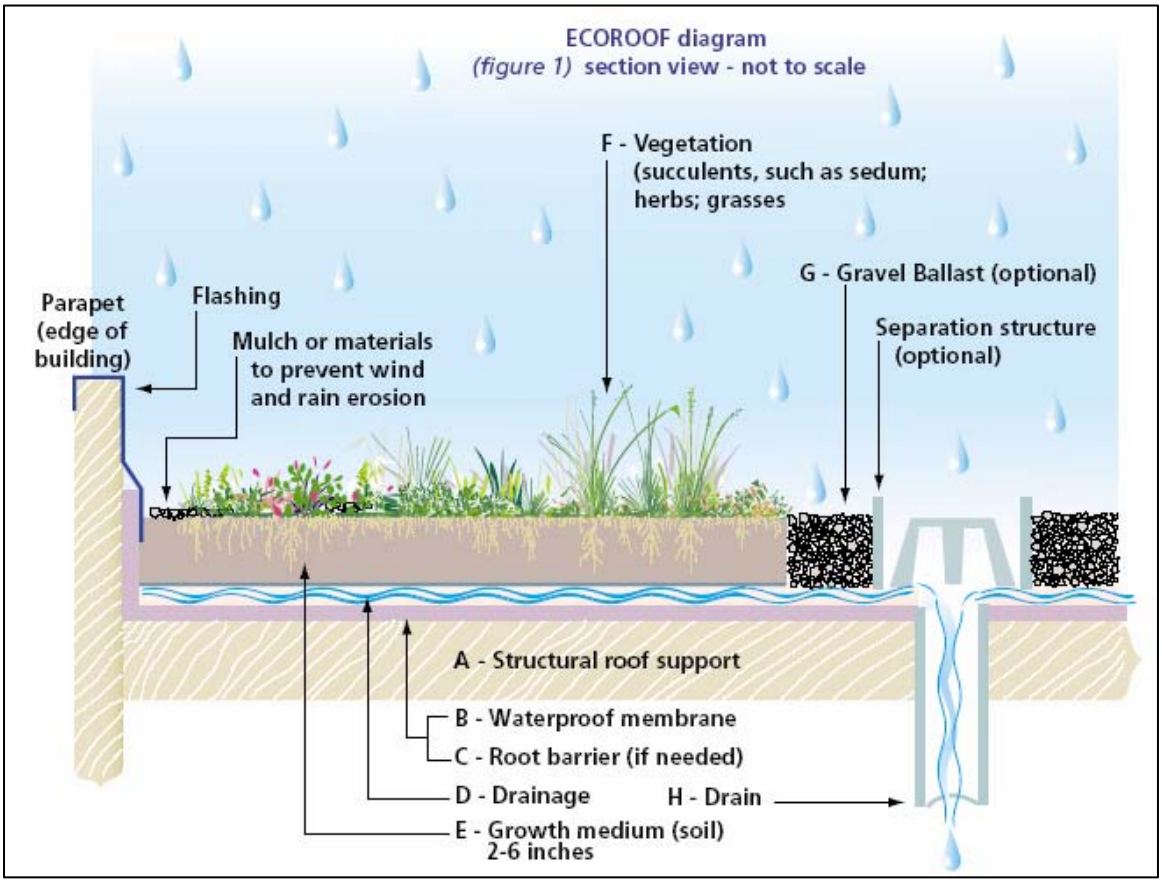


Figure 5.2.22-1 Green Roof Cross Section (from City of Portland, Oregon)

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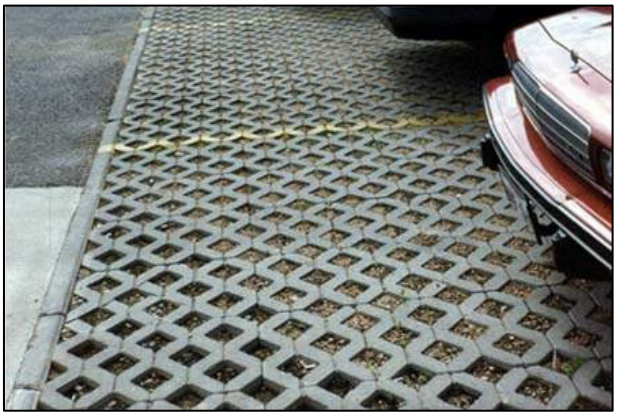
5.2.23 Modular Porous Paver Systems

Structural Storm Water Control



Description: A pavement surface composed of structural units with void areas that are filled with pervious materials such as sand or grass turf. Porous pavers are installed over a gravel base course that provides storage as runoff infiltrates through the porous paver system into underlying permeable soils.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for low traffic areas, or for residential or overflow parking applications
- Soil infiltration rate of 0.5 in/hr or greater required

ADVANTAGES / BENEFITS:

- Provides reduction in runoff volume
- High level of pollutant removal
- Available from commercial vendors

DISADVANTAGES / LIMITATIONS:

- High maintenance requirements
- High cost compared to conventional pavements
- Potential for high failure rate if not adequately maintained or used in unstabilized areas
- Potential for groundwater contamination

POLLUTANT REMOVAL

NA	**Total Suspended Solids**
80/80%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
90%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [S] **Water Quality Protection**
- [S] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [L] **Land Requirement**
- [M] **Capital Cost**
- [H] **Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *No restrictions.*

Soils: *Soil infiltration rate of 0.5 in/hr or greater required*

L=Low **M**=Moderate **H**=High

5.2.23.1 General Description

Modular porous pavers are structural units, such as concrete blocks, bricks, or reinforced plastic mats, with regularly interspersed void areas used to create a load-bearing pavement surface. The void areas are filled with pervious materials (gravel, sand, or grass turf) to create a system that allows for the infiltration of storm water runoff. Porous paver systems provide water quality benefits in addition to groundwater recharge and a reduction in storm water volume. The use of porous paver systems results in a reduction of the effective impervious area on a site.

There are many different types of modular porous pavers available from different manufacturers, including both pre-cast and mold in-place concrete blocks, concrete grids, interlocking bricks, and plastic mats with hollow rings or hexagonal cells (see Figure 5.2.23-1).

Modular porous pavers are typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the porous paver surface into the gravel base course, which acts as a storage reservoir as it infiltrates to the underlying soil. The infiltration rate of the soils in the subgrade must be adequate to support drawdown of the entire runoff capture volume within 24 to 48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

Modular porous paver systems are typically used in low-traffic areas such as the following types of applications:

- Parking pads in parking lots
- Overflow parking areas
- Residential driveways
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes

A major drawback is the cost and complexity of modular porous paver systems compared to conventional pavements. Porous paver systems require a very high level of construction workmanship to ensure that they function as designed and do not settle unevenly. In addition, there is the difficulty and cost of rehabilitating the surfaces should they become clogged. Therefore, consideration of porous paver systems should include the construction and maintenance requirements and costs.

5.2.23.2 Pollutant Removal Capabilities

As they provide for the infiltration of storm water runoff, porous paver systems have a high removal of both soluble and particulate pollutants, where they become trapped, absorbed, or broken down in the underlying soil layers. Due to the potential for clogging, porous paver surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- **Total Suspended Solids – not applicable**
- **Total Phosphorus – 80%**
- **Total Nitrogen – 80%**
- **Fecal Coliform – insufficient data**

- **Heavy Metals – 90%**

5.2.23.3 Design Criteria and Specifications

- Porous paver systems can be used where the underlying in-situ subsoils have an infiltration rate of between 0.5 and 3.0 inches per hour. Therefore, porous paver systems are not suitable on sites with hydrologic group D or most group C soils, or soils with a high (>30%) clay content. During construction and preparation of the subgrade, special care must be taken to avoid compaction of the soils.
- Porous paver systems should ideally be used in applications where the pavement receives tributary runoff only from impervious areas. The ratio of the contributing impervious area to the porous paver surface area should be no greater than 3:1.
- If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the porous paver surface.
- Porous paver systems are not recommended on sites with a slope greater than 2%.
- A minimum of 2 feet of clearance is required between the bottom of the gravel base course and underlying bedrock or the seasonally high groundwater table.
- Porous paver systems should be sited at least 10 feet downgradient from buildings and 100 feet away from drinking water wells.
- An appropriate modular porous paver should be selected for the intended application. A minimum of 40% of the surface area should consist of open void space. If it is a load bearing surface, then the pavers should be able to support the maximum load.
- The porous paver infill is selected based upon the intended application and required infiltration rate. Masonry sand (such as ASTM C-33 concrete sand or TxDOT item 421 Fine Aggregate) has a high infiltration rate (8 in/hr) and should be used in applications where no vegetation is desired. A sandy loam soil has a substantially lower infiltration rate (1 in/hr), but will provide for growth of a grass ground cover.
- A 1-inch top course (filter layer) of sand (ASTM C-33 concrete sand or TxDOT item 421 Fine Aggregate) underlain by filter fabric is placed under the porous pavers and above the gravel base course.
- The gravel base course should be designed to store at a minimum the water quality protection volume (WQ_v). The stone aggregate used should be washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40% (ASTM C-33 Size No. 3 Coarse Aggregate). Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations.
- The gravel base course must have a minimum depth of 9 inches. The following equation can be used to determine if the depth of the storage layer (gravel base course) needs to be greater than the minimum depth:

  $$d = V / A * n$$

  where:
  - d = Gravel Layer Depth (feet)
  - V = Water Quality Protection Volume –or– Total Volume to be Infiltrated (cubic feet)
  - A = Surface Area (square feet)
  - n = Porosity (use n=0.32)

- The surface of the subgrade should be lined with filter fabric or an 8-inch layer of sand (ASTM C-33 concrete sand or TxDOT item 421 Fine Aggregate) and be completely flat to promote infiltration across the entire surface.

- Porous paver system designs must use some method to convey larger storm event flows to the conveyance system. One option is to use storm drain inlets set slightly above the elevation of the pavement. This would allow for some ponding above the surface, but would accept bypass flows that are too large to be infiltrated by the porous paver system, or if the surface clogs.
- For the purpose of sizing downstream conveyance and structural control system, porous paver surface areas can be assumed to be 35% impervious. In addition, a reduction in water quality volume requirements can be obtained for the runoff volume infiltrated from other impervious areas using the methodology in Section 1.2.

5.2.23.4 Inspection and Maintenance Requirements

Table 5.2.23-1 Typical Maintenance Activities for Modular Porous Paver Systems

Activity	Schedule
• Ensure that the porous paver surface is free of extraneous sediment. • Check to make sure that the system dewaters between storms.	Monthly
• Clear debris from contributing area and porous paver surface. • Stabilize and mow contributing adjacent areas and remove clippings.	As needed, based on inspection
• Vacuum sweep porous paver surface to keep free of sediment.	Typically three to four times a year
• Inspect the surface for deterioration or spalling.	Annually
• Totally rehabilitate the porous paver system, including the top and base course.	Upon failure

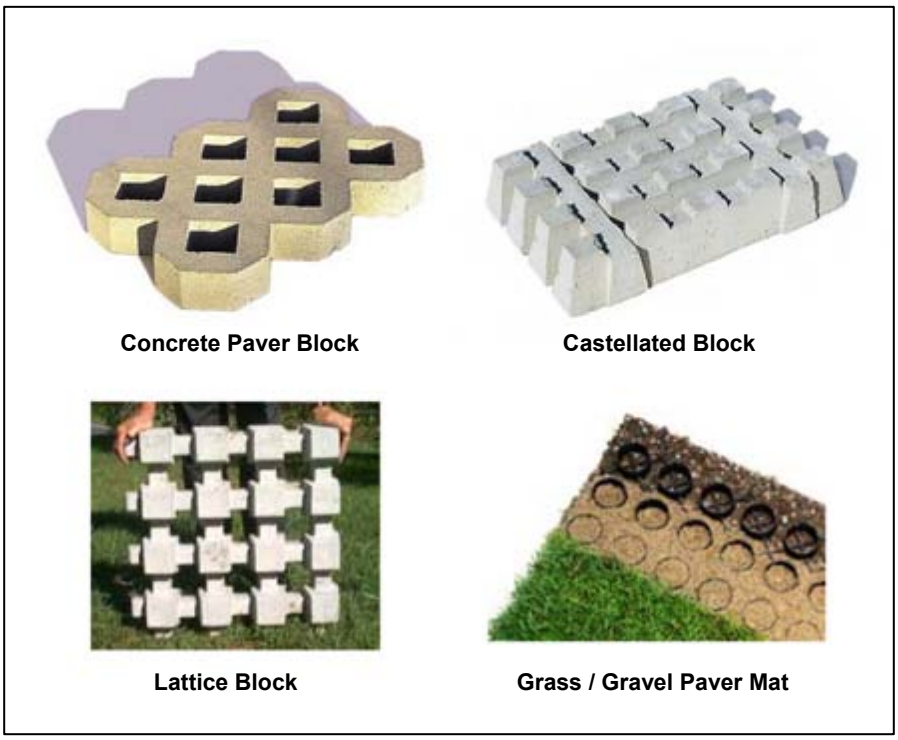


Figure 5.2.23-1 Examples of Modular Porous Pavers

5.2.23.5 Example Schematics

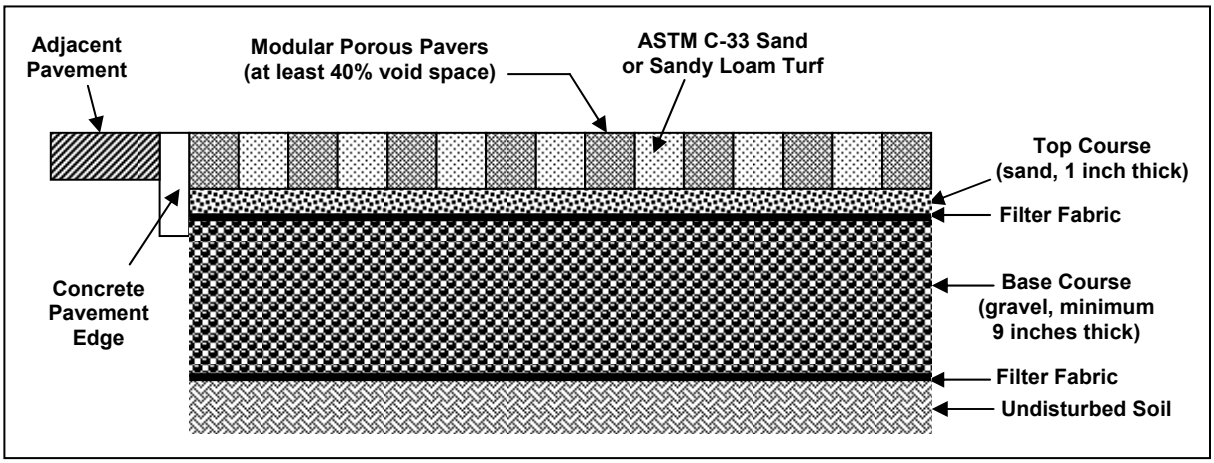


Figure 5.2.23-2 Modular Porous Paver System Section

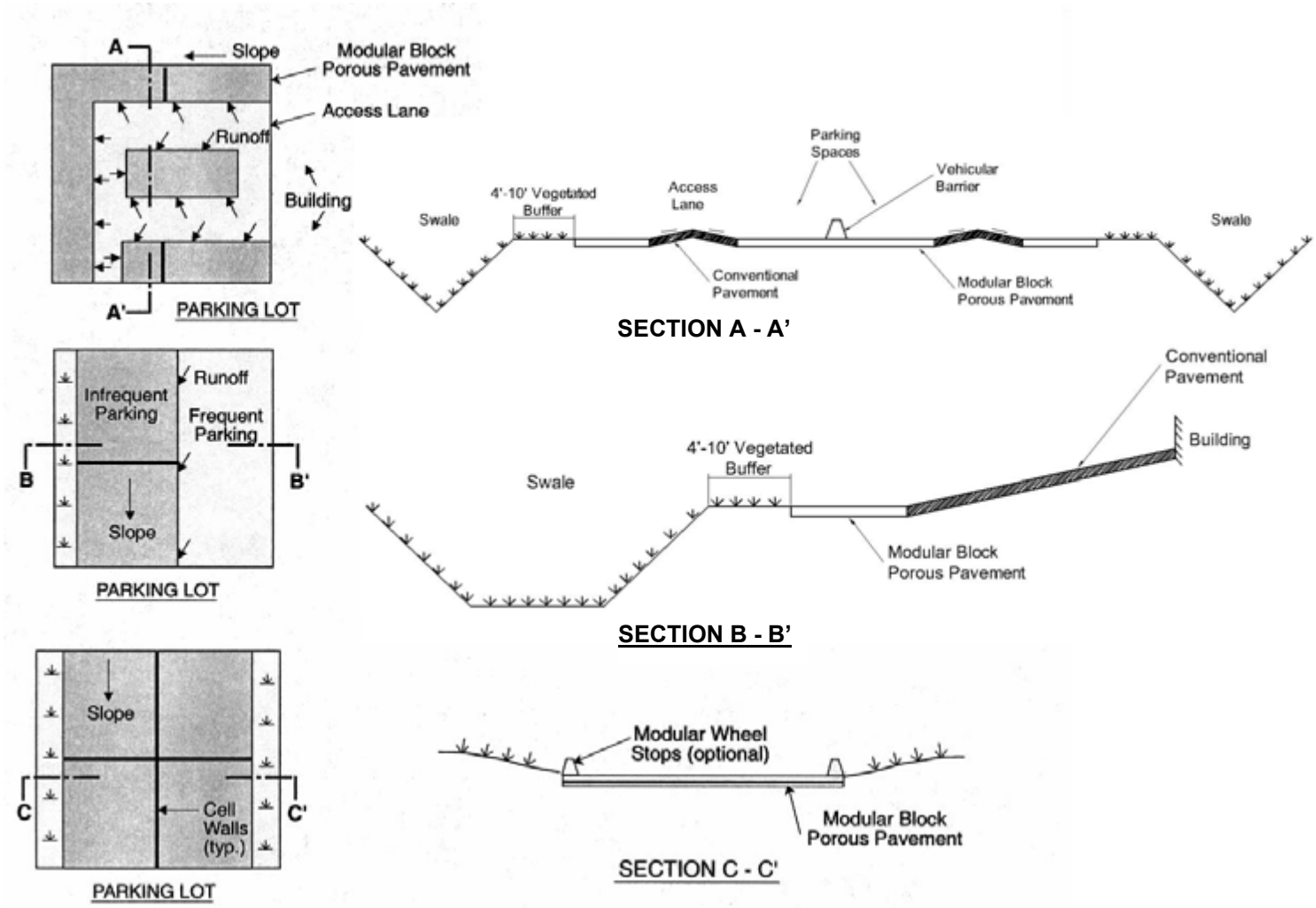
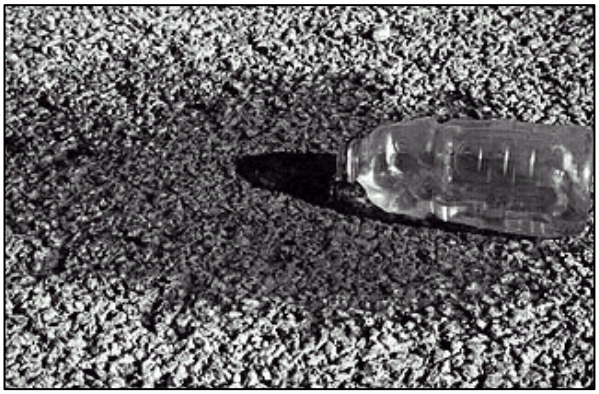


Figure 5.2.23-3 Typical Modular Porous Paver System Applications
(Source: UDFCD, 1999)

Figure 5.2.23-4 Examples of Porous Paver Surfaces
(Sources: Invisible Structures, Inc.; EP Henry Corp.)

5.2.24 Porous Concrete

Limited Application
Structural Storm Water Control



Description: Porous concrete is the term for a mixture of coarse aggregate, portland cement and water that allows for rapid infiltration of water and overlays a stone aggregate reservoir. This reservoir provides temporary storage as runoff infiltrates into underlying permeable soils and/or out through an underdrain system.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for low volume auto traffic areas, or for overflow parking applications
- Should not be used in areas of soils with low permeability, wellhead protection zones, or recharge areas of water supply aquifer recharge areas.
- Soil infiltration rate of 0.5 in/hr or greater required
- Excavated area filled with stone media; gravel and sand filter layers with observation well

ADVANTAGES / BENEFITS:

- Pre-treat runoff if sediment present
- Provides reduction in runoff volume

DISADVANTAGES / LIMITATIONS:

- Restrictions on use by heavy vehicles
- High maintenance requirements
- Special attention to design and construction needed
- Somewhat higher cost when compared to conventional pavements
- Potential for high failure rate if poorly designed, poorly constructed, not adequately maintained, or used in unstabilized areas
- Potential for groundwater contamination

POLLUTANT REMOVAL

NA	**Total Suspended Solids**
50/65%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
60%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

- [S] **Water Quality Protection**
- [S] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [L] **Land Requirement**
- [H] **Capital Cost**
- [H] **Maintenance Burden**

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

Other Considerations:

- *Overflow Parking, driveway*

L=Low **M**=Moderate **H**=High

5.2.24.1 General Description

Porous concrete (also referred to as *enhanced porosity concrete*, *porous concrete*, *portland cement pervious pavement,* and *pervious pavement*) is a subset of a broader family of pervious pavements including porous asphalt, and various kinds of grids and paver systems. Porous concrete is thought to have a greater ability than porous asphalt to maintain its porosity in hot weather and thus is provided as a limited application control in this manual. Although, porous concrete has seen growing use, there is still very limited practical experience with this measure. According to the U.S. EPA, porous pavement sites have had a high failure rate – approximately 75 percent. Failure has been attributed to poor design, inadequate construction techniques, soils with low permeability, heavy vehicular traffic, and poor maintenance. This measure, if used, should be carefully monitored over the life of the development.

Porous concrete consists of a specially formulated mixture of portland cement; uniform, open graded course aggregate; and water. The concrete layer has a high permeability often many times that of the underlying permeable soil layer which allows rapid percolation of rainwater through the surface and into the layers beneath. The void space in porous concrete is in the 15% to 22% range compared to three to five percent for conventional pavements. The permeable surface is placed over a layer of open-graded gravel and crushed stone. The void spaces in the stone act as a storage reservoir for runoff.

Porous concrete is designed primarily for storm water quality, i.e. the removal of storm water pollutants. However, it can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the streambank protection volume (SP_v) in addition to WQ_v. Porous concrete will need to be used in conjunction with another structural control to provide downstream flood control, if required.

Modifications or additions to the standard design have been used to pass flows and volumes in excess of the water quality volume, or to increase storage capacity or treatment. These include:

- Placing a perforated pipe near the top of the crushed stone reservoir to pass excess flows after the reservoir is filled
- Providing surface detention storage in a parking lot, adjacent swale, or detention pond with suitable overflow conveyance
- Connecting the stone reservoir layer to a stone filled trench
- Adding a sand layer and perforated pipe beneath the stone layer for filtration of the water quality volume
- Placing an underground detention tank or vault system beneath the layers

The infiltration rate of the soils in the subgrade should be adequate to support drawdown of the entire runoff capture volume within 24 to 48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils which could affect the soils' infiltration capability.

Porous concrete systems are typically used in low-traffic areas such as the following types of applications:

- Parking pads in parking lots
- Overflow parking areas
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes

Slopes should be flat or gentle to facilitate infiltration versus runoff and the seasonally high water table or bedrock should be a minimum of two feet below the bottom of the gravel layer if infiltration is to be relied on to remove the stored volume.

Porous concrete has the positive characteristics of volume reduction due to infiltration, groundwater recharge, and an ability to blend into the normal urban landscape relatively unnoticed. It also allows a reduction in the cost of other storm water infrastructure, a fact that may offset the greater placement cost somewhat.

A drawback is the cost and complexity of porous concrete systems compared to conventional pavements. Porous concrete systems require a very high level of construction workmanship to ensure that they function as designed. They experience a high failure rate if they are not designed, constructed, and maintained properly.

Like other infiltration controls, porous concrete should not be used in areas that experience high rates of wind erosion, where highly erosive soils are present, or in drinking water aquifer recharge areas. Also it cannot be used in traffic areas where sanding is used during winter weather.

5.2.24.2 Pollutant Removal Capabilities

As they provide for the infiltration of storm water runoff, porous concrete systems have a high removal of both soluble and particulate pollutants. These pollutants become trapped, absorbed, or broken down in the underlying soil layers. Due to the potential for clogging, porous concrete surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- **Total Suspended Solids – not applicable**
- **Total Phosphorus – 50%**
- **Total Nitrogen – 65%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 60%**

Pollutant removal can be improved through routine vacuum sweeping and high pressure washing, insuring a drainage time of at least 24 hours, pretreating the runoff, having organic material in the subsoil, and using clean washed aggregate (EPA, 1999).

5.2.24.3 Design Criteria and Specifications

- Porous concrete systems can be used where the underlying in-situ subsoils have an infiltration rate greater than 0.5 inches per hour. Therefore, porous concrete systems are not suitable on sites with hydrologic group D or most group C soils, or soils with a high (>30%) clay content. During construction and preparation of the subgrade, special care must be taken to avoid compaction of the underlying soils.
- Porous concrete systems should typically be used in applications where the pavement receives tributary runoff only from impervious areas. Actual pervious surface area sizing will depend on achieving a 24 hour minimum and 48 hour maximum draw down time for the design storm volume.
- If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the porous paver surface. Pretreatment using filter strips or vegetated swales for removal of course sediments is recommended. (see sections 5.2.12 and 5.2.3)

- Porous concrete systems should not be used on slopes greater than 5% with slopes of no greater than 2% recommended. For slopes greater than 1% barriers perpendicular to the direction of drainage should be installed in sub-grade material to keep it from washing away, or filter fabric should be placed at the bottom and sides of the aggregate to keep soil from migrating into the aggregate and reducing porosity.
- A minimum of four feet of clearance is recommended between the bottom of the gravel base course and underlying bedrock or the seasonally high groundwater table.
- Porous concrete systems should be sited at least 10 feet down-gradient from buildings and 100 feet away from drinking water wells.
- To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Porous concrete should not be used for manufacturing and industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals. In addition, porous concrete should not be considered for areas with a high pesticide concentration. Porous concrete is also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements.
- Porous concrete system designs must use some method to convey larger storm event flows to the conveyance system. One option is to use storm drain inlets set slightly above the elevation of the pavement. This would allow for some ponding above the surface, but would accept bypass flows that are too large to be infiltrated by the porous concrete system, or if the surface clogs.
- For the purpose of sizing downstream conveyance and structural control system, porous concrete surface areas can be assumed to 35% impervious. In addition, reduction in water quality volume requirements can be obtained for the runoff volume infiltrated from other impervious areas using the methodology in Section 1.2.
- For treatment control, the design volume should be, at a minimum, equal to the water quality volume. The water quality storage volume is contained in the surface layer, the aggregate reservoir, and the sub-grade above the seasonal high water table – if the sub-grade is sandy. The storm duration (fill time) is normally short compared to the infiltration rate of the sub-grade, a duration of <u>two hours</u> can be used for design purposes. The total storage volume in a layer is equal to the percent of voids times the volume of the layer. Alternately storage may be created on the surface through temporary ponding, though this would tend to accelerate clogging if course sediment or mud settles out on the surface.
- The cross-section typically consists of four layers, as shown in Figure 5.2.24-1. The aggregate reservoir can sometimes be avoided or minimized if the sub-grade is sandy and there is adequate time to infiltrate the necessary runoff volume into the sandy soil without by-passing the water quality volume. Descriptions of each of the layers is presented below:

  <u>*Porous Concrete Layer*</u> – The porous concrete layer consists of an open-graded concrete mixture usually ranging from depths of 2 to 4 inches depending on required bearing strength and pavement design requirements. Porous concrete can be assumed to contain 18 percent voids (porosity = 0.18) for design purposes. Thus, for example, a 4 inch thick porous concrete layer would hold 0.72 inches of rainfall. The omission of the fine aggregate provides the porosity of the porous pavement. To provide a smooth riding surface and to enhance handling and placement a coarse aggregate of 3/8 inch maximum size is normally used. Use coarse aggregate (3/8 to No. 16) per ASTM C 33 or No. 89 coarse aggregate (3/8 to No. 50) per ASTM D 448.

  <u>*Top Filter Layer*</u> – Consists of a 0.5 inch diameter crushed stone to a depth of 1 to 2 inches. This layer serves to stabilize the porous concrete layer. It can be combined with reservoir layer using suitable stone.

  <u>*Reservoir Layer*</u> – The reservoir gravel base course consists of washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40% meeting the gradation listed below. The depth of this layer depends on the desired storage volume, which is a function of the soil infiltration rate and void spaces, but typically ranges from two to four feet. The layer must have a minimum depth of nine

inches. The layer should be designed to drain completely in 48 hours. and stored at a minimum the water quality volume (WQ_v). Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations unless aggregate specific data exist.

Gradation

Sieve Size	% Passing
2 ½"	100
2"	90 – 100
1 ½"	35 – 70
1"	0 – 15
½"	0 - 5

Bottom Filter Layer – The surface of the subgrade should be an 6 inch layer of sand (ASTM C-33 concrete sand or TxDOT Fine Aggregate Grade No. 1) or a 2 inch thick layer of 0.5 inch crushed stone, and be completely flat to promote infiltration across the entire surface. This layer serves to stabilize the reservoir layer, to protect the underlying soil from compaction, and act as the interface between the reservoir layer and the filter fabric covering the underlying soil.

Filter Fabric – It is very important to line the entire trench area, including the sides, with filter fabric prior to placement of the aggregate. The filter fabric serves a very important function by inhibiting soil from migrating into the reservoir layer and reducing storage capacity. Fabric should be MIRIFI # 14 N or equivalent.

Underlying Soil – The underlying soil should have an infiltration capacity of at least 0.5 in/hr, but preferably greater than 0.50 in/hr. as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils. Soils at the lower end of this range may not be suited for a full infiltration system. Test borings are recommended to determine the soil classification, seasonal high ground water table elevation, and impervious substrata, and an initial estimate of permeability. Often a double-ring infiltrometer test is done at subgrade elevation to determine the infiltration rate of the least permeable layer, and, for safety, one-half that measured value is taken for infiltration calculations.

- The pit excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench as not to jeopardize the stability of the trench sidewalls. The bottom of the excavated trench should not be loaded so as to cause compaction, and should be scarified prior to placement of sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction, and should be constructed after upstream areas have been stabilized.
- An observation well consisting of perforated PVC pipe 4 to 6 inches in diameter should be placed at the downstream end of the facility and protected. The well should be used to determine actual infiltration rates.
- A warning sign should be placed at the facility that states, "Porous Paving used on this site to reduce pollution. Do not resurface with non-porous material or sand during icy weather. Call the local jurisdiction for more information."
- Details of construction of the concrete layer are beyond the scope of this manual. However, construction of porous concrete is exacting, and requires special handling, timing, and placement to perform adequately (LACDPW, 2000, Paine, 1992, Maryland, 1984).

5.2.24.4 Inspection and Maintenance Requirements

Table 5.2.24-1 Typical Maintenance Activities for Porous Concrete Systems

Activity	Schedule
• Initial inspection	Monthly for three months after installation
• Ensure that the porous paver surface is free of sediment	Monthly
• Ensure that the contributing and adjacent area is stabilized and mowed, with clippings removed	As needed, based on inspection
• Vacuum sweep porous concrete surface followed by high pressure hosing to keep pores free of sediment	Four times a year
• Inspect the surface for deterioration or spalling • Check to make sure that the system dewaters between storms	Annually
• Spot clogging can be handled by drilling half-inch holes through the pavement every few feet • Rehabilitation of the porous concrete system, including the top and base course as needed	Upon failure

To ensure proper maintenance of porous pavement, a carefully worded maintenance agreement is essential. It should include specific the specific requirements and establish the responsibilities of the property owner and provide for enforcement.

5.2.24.5 Example Schematics

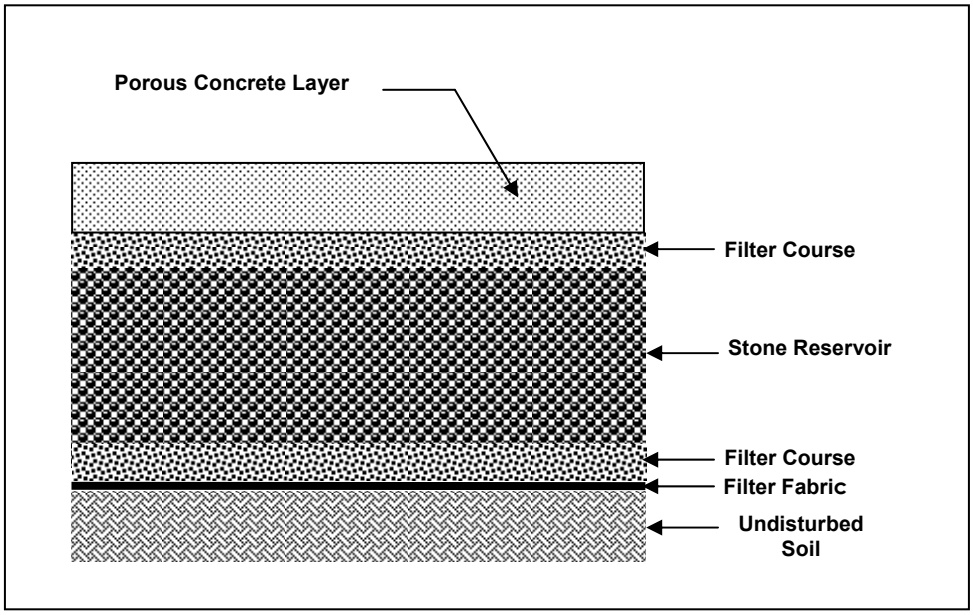


Figure 5.2.24-1 Porous Concrete System Section
(Modified From: LAC 2000)

Figure 5.2.24-2 Porous Concrete System Installation

Figure 5.2.24-3 Typical Porous Concrete System Applications
(Photos by Bruce Ferguson, Don Wade)

5.2.24.6 Design Example

Data

A 1.5 acre overflow parking area is to be designed to provide water quality treatment using porous concrete to handle the runoff from the whole overflow parking area. Initial data shows:

- Rainfall depth for treatment is up to 1.5 inches
- Borings show depth to water table is 5.0 feet
- Boring and infiltrometer tests show sand-loam with percolation rate (k) of 1.02 inches/hr

- Structural design indicates the thickness of the porous concrete must be at least three inches

Water Quality Volume

R_v = 0.05 + 0.009 I (where I = 100 percent)
= 0.95

WQ_v = 1.5 R_v A / 12 = 1.5 * 0.95 * 1.5/12 = 0.178 acre-feet
= (0.178 ac-ft) (43,560 cu-ft/ac-ft) = 7,759 cubic feet

Surface Area

A porosity value n = 0.32 should be used for the gravel and 0.18 for the concrete layer.

All infiltration systems should be designed to fully de-water the entire WQ_v within 24 to 48 hours after the rainfall event at the design percolation rate.

A fill time T=2 hours can be used for most designs

Chose a depth of gravel pit of three feet (including layer under concrete) which fits the site with a two foot minimum to water table (other lesser depths could be chosen, making the surface area larger). The minimum surface area of the trench can be determined, in a manner similar to the infiltration trench, from the following equation:

$A = WQ_v/(n_g d_g + kT/12 + n_p d_p)$

= 7,759/(0.32*3 + 1.02*2/12 + 0.18 * 3/12)

= 6,604 square feet

Where:
A = Surface Area
WQv = Water Quality Volume (or total volume to be infiltrated)
n = porosity (g of the gravel, p of the concrete layer)
d = depth or gravel layer (feet) (g of the gravel, p of the concrete layer)
k = percolation (inches/hour)
T = Fill Time (time for the practice to fill with water), in hours

Check of drain time:
depth = 3*12 + 3 inches to sand layer = 39 inches @ 1.02 in/hr = 38 hours (ok)

Overflow will be carried across the porous concrete and tied into the drainage system for the rest of the site.

5.2.25 Proprietary Structural Controls

Limited Application
Structural Storm Water Control

Description: Manufactured structural control systems available from commercial vendors designed to treat storm water runoff and/or provide water quantity control

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Independent performance data must be available to prove a demonstrated capability of meeting storm water management goal(s)
- System or device must be appropriate for use in North Central Texas conditions, and specifically for the community in question
- Pre-treat runoff if sediment present

ADVANTAGES / BENEFITS:

- Provides reduction in runoff volume

DISADVANTAGES / LIMITATIONS:

- Depending on the proprietary system, there may be:
  - Limited performance data
  - Application constraints
  - High maintenance requirements
  - Higher costs than other structural control alternatives
- Installation and operations/maintenance requirements must be understood by all parties approving and using the system or device in question

STORM WATER MANAGEMENT SUITABILITY

- S **Water Quality Protection**
- S **Streambank Protection**
- S **On-Site Flood Control**
- S **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- L **Land Requirement**
- H **Capital Cost**
- H **Maintenance Burden**

Residential Subdivision Use: *Depends on the specific proprietary structural control*

High Density/Ultra-Urban: *Yes*

Drainage Area: *Depends on the specific proprietary structural control.*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

Note: It is the policy of this Manual not to recommend any specific commercial vendors for proprietary systems. However, this subsection is being included in order to provide communities with a rationale for approving the use of a proprietary system or practice in their jurisdictions.

5.2.25.1 General Description

There are many types of commercially-available proprietary storm water structural controls available for both water quality treatment and quantity control. These systems include:

- Hydrodynamic systems such as gravity and vortex separators
- Filtration systems
- Catch basin media inserts
- Chemical treatment systems

- Package treatment plants
- Prefabricated detention structures

Many proprietary systems are useful on small sites and space-limited areas where there is not enough land or room for other structural control alternatives. Proprietary systems can often be used in pretreatment applications in a treatment train. However, proprietary systems are often more costly than other alternatives and may have high maintenance requirements. Perhaps the largest difficulty in using a proprietary system is the lack of adequate independent performance data, particularly for use in North Central Texas conditions. Below are general guidelines that should be followed before considering the use of a proprietary commercial system.

5.2.25.2 Guidelines for Using Proprietary Systems

In order for use as a limited application control, a proprietary system must have a demonstrated capability of meeting the storm water management goals for which it is being intended. This means that the system must provide:

1. Independent third-party scientific verification of the ability of the proprietary system to meet water quality treatment objectives and/or to provide water quantity control (streambank or flood control)
2. Proven record of longevity in the field
3. Proven ability to function in North Central Texas conditions (e.g., climate, rainfall patterns, soil types, etc.)
4. Maintainability - Documented procedures for required maintenance including collection and removal of pollutants or debris.

For a proprietary system to meet (1) above for water quality goals, the following monitoring criteria should be met for supporting studies:

- At least 15 storm events must be sampled
- The study must be independent or independently verified (i.e., may not be conducted by the vendor or designer without third-party verification)
- The study must be conducted in the field, as opposed to laboratory testing
- Field monitoring must be conducted using standard protocols which require proportional sampling both upstream and downstream of the device
- Concentrations reported in the study must be flow-weighted
- The propriety system or device must have been in place for at least one year at the time of monitoring

Although local data is preferred, data from other regions can be accepted as long as the design accounts for the local conditions.

Local governments may submit a proprietary system to further scrutiny based on the performance of similar practices. A poor performance record or high failure rate is valid justification for not allowing the use of a proprietary system or device. Consult your local review authority for more information in regards to the use of proprietary structural storm water controls.

5.2.26 Rain Harvesting (Tanks/Barrels)

Storm Water Control

Description: Rain harvesting is a container or system designed to capture and store rainwater discharged from a roof. The rain harvesting system consists of a storage container, a downspout diversion, a sealed lid, and an overflow system. Typical rain harvesting systems hold between 50 and 500 gallons of water, and may work in series to provide larger volumes of storage.

KEY CONSIDERATIONS

ADVANTAGES / BENEFITS:

- Provides reduction in runoff volume
- Low-cost, effective, and easy to maintain
- Offers flexibility with volume of water to capture
- Potential water savings
- Healthier for plants and gardens due to non-chlorinated water

DISADVANTAGES / LIMITATIONS:

- Small storage capacity
- Requires some attention
- If not attended to after a rain, leaking can cause damage to adjacent building foundation
- High construction cost when compared to the low cost of municipal water supply
- Certain roofing materials can cause runoff contamination (re-use)

STORM WATER MANAGEMENT SUITABILITY

- [P] **Water Quality Protection**
- [ ] **Streambank Protection**
- [ ] **On-Site Flood Control**
- [ ] **Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

- [L] **Land Requirement**
- [L] **Capital Cost**
- [H] **Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *depends on manufacture's model*

Soils: *No restrictions*

L=Low **M**=Moderate **H**=High

5.2.26.1 General Description

Rain harvesting (also referred to as *rain pails* and *rain savers*) are used as a water conservation practice and a storm water management strategy. Capturing water in a rain tank/barrel prevents runoff from flowing down a driveway or across a parking lot and picking up soil, pesticides, and other pollutants before entering the storm sewer system.

Rain tanks/barrels not only store water, but also help to decrease the water supply demand during the sweltering summer months. Only 1/4 inch of rainfall runoff from the average roof will completely fill the typical barrel. Collection of water from rooftop runoff can provide an ample supply of free 'soft water' containing no chlorine, lime, or calcium. Because it tends to have fewer sediments and dissolved salts than municipal water, rain water is ideal for a multitude of applications, including organic vegetable gardens, planter beds for botanicals, indoor plants, automobile washing, and cleaning household windows. Saving water in this manner will reduce the demand for treated tap water, and save money by lowering the homeowner's monthly bill. Rain water diversion will also help decrease the burden on water treatment facilities and municipal drainage systems during storm events.

A typical rain harvesting design will include a storage container with a hole at the top to allow for flow from a downspout, a sealed lid, an overflow pipe and a spigot at or near the bottom of the barrel. The spigot can be left partially open to detain water or closed to fill the barrel. A screen is often included to control mosquitoes and other insects. Rain tanks/barrels can be connected in series to provide larger volumes of storage. Larger systems for commercial or industrial use can include pumps and filtration systems.

For every inch of rain that falls on a catchment area of 1,000 square feet, approximately 600 gallons of rainwater can be collected. Ten inches of rain falling on a 1,000 square foot catchment area will generate approximately 6,000 gallons of rainwater.

5.2.26.2 Design Criteria and Specifications

- The required capacity of a rain barrel is a function of the rooftop surface area that drains to it, the inches of rainfall required to fill the barrel, and water losses due primarily to evaporation. The general rule of thumb to utilize in the sizing of rain barrels is 1 inch of rainfall on a 1000 square foot roof will yield approximately 600 gallons.

Sample Calculation		
Rain barrel volume can be determined by calculating the roof top water yield for any given rainfall, using the following general equation:		
Equation 1.	$V = A^2 \times R \times 0.90 \times 7.5$ *gals./ ft.*^{3}	where:
	V	= volume of rain barrel (gallons)
	A^2	= surface area roof (square feet)
	R	= rainfall (feet)
	0.90	= losses to system (no units)
	7.5	= conversion factor (gallons per cubic foot)
Example: one 60-gallon barrel would provide runoff storage from a rooftop area of approximately 212 square feet for a 1.5 inch (0.125 ft.) of rainfall. $V = 212\ ft.^2 \times 0.125\ ft. \times 0.90 \times 7.5\ gallons/ft.^3 = 179\ gallons$		

- Homeowners and manufacturers typically use a flexible plastic downspout elbow to direct water from the downspout into the rain tank/barrel. It is best to use the downspout connector only for smaller drainage areas or to use a diverter that can be engaged either automatically or with physical contact.
- The use of screens should be considered on gutters and downspouts to remove sediment and particles as the water enters the barrel.
- Containers should be opaque to discourage bacteria/algae growth.
- Most rain barrels are equipped with tight covers and screens to prevent accidents involving children and pets and to keep debris out of the barrels. To combat concerns regarding mosquitoes and West Nile virus, tight fitting screens should be inspected and maintained on a routine basis or nontoxic mosquito control agent (Bti) placed in water.
- A half-barrel design will allow the barrel to sit flush against a building and may prove to be more aesthetically pleasing.
- A typical rain barrel will include spigot at the top to accommodate overflow (this should be directed away from the foundation of the building) and a gravity-fed hose bib at the bottom to connect a hose for redistribution of the rainwater.
- Inexpensive rain barrels can be made from food grade plastic barrels or heavy-duty trash cans, for as little as $15 or they can be purchased pre-made from numerous non-profit organizations, commercial manufacturers, and retailers, in prices ranging from $25 to $150.

5.2.26.3 Example Schematics

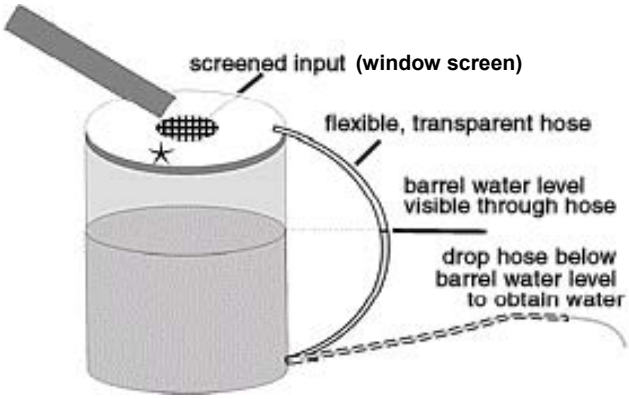


Figure 5.2.26-1 Simple Rain Barrel Design (from Maryland DNR Green Building Program)

Figure 5.2.26-2 Rain barrels in series

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5.2.27 Storm Water Wetlands

Storm Water Control

Description: Constructed wetland systems used for storm water management. Runoff volume is both stored and treated in the wetland facility.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum contributing drainage area of 25 acres; 5 acres for pocket wetland
- Minimum dry weather flow path of 2:1 (length: width) should be provided from inflow to outflow
- Minimum of 35% of total surface area should have a depth of 6 inches or less; 10 to 20% of surface area should be deep pool (1.5- to 6-foot depth)

ADVANTAGES / BENEFITS:

- Good nutrient removal
- Provides natural wildlife habitat
- Relatively low maintenance costs

DISADVANTAGES / LIMITATIONS:

- Requires large land area
- Needs continuous baseflow for viable wetland
- Sediment regulation is critical to sustain wetlands
- Large commitment to establish vegetation in the first 3 years

MAINTENANCE REQUIREMENTS:

- Replace wetland vegetation to maintain at least 50% surface area coverage
- Remove invasive vegetation
- Monitor sediment accumulation and remove periodically

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
40/30%	***Nutrients -** Total Phosphorus / Total Nitrogen removal
50%	**Metals -** Cadmium, Copper, Lead, and Zinc removal
70%	**Pathogens -** Coliform, Streptococci, E.Coli removal

STORM WATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
P	**Streambank Protection**
P	**On-Site Flood Control***
P	**Downstream Flood Control***

**Does not apply to Submerged Gravel Wetland Systems*

Accepts Hotspot Runoff: *Yes*
(2 feet of separation distance required to water table)

IMPLEMENTATION CONSIDERATIONS

M-H	**Land Requirement**
M	**Capital Cost**
	Maintenance Burden:
M	Shallow Wetland
M	ED Shallow Wetland
H	Pocket Wetland
M	Pond/Wetland

Residential Subdivision Use: *Yes*

High-Density/Ultra-Urban: *No*

Drainage Area: *25 acres min.*

Soils: *Hydrologic group 'A' and 'B' soils may require a liner*

L=Low M=Moderate H=High

5.2.27.1 General Description

Storm water wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban storm water and control runoff volumes. As storm water runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

Wetlands are among the most effective storm water practices in terms of pollutant removal and also offer aesthetic value and wildlife habitat. Constructed storm water wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for the purpose of treating storm water runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, storm water wetlands require a continuous base flow or a high water table to support aquatic vegetation.

There are several design variations of the storm water wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system, and pocket wetland. Figure 5.2.27-1 contains photos of various wetlands. Below are descriptions of each design variant:

- **Shallow Wetland** – In the shallow wetland design, most of the water quality treatment volume is in the relatively shallow high marsh or low marsh depths. The only deep portions of the shallow wetland design are the forebay at the inlet to the wetland, and the micropool at the outlet. One disadvantage of this design is that, since the pool is very shallow, a relatively large amount of land is typically needed to store the water quality volume.
- **Extended Detention (ED) Shallow Wetland** – The extended detention (ED) shallow wetland design is the same as the shallow wetland; however, part of the water quality treatment volume is provided as extended detention above the surface of the marsh and released over a period of 24 hours. This design can treat a greater volume of storm water in a smaller space than the shallow wetland design. In the extended detention wetland option, plants that can tolerate both wet and dry periods need to be specified in the extended detention zone.
- **Pond/Wetland Systems** – The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediments and reduces runoff velocities prior to entry into the wetland where storm water flows receive additional treatment. Less land is required for a pond/wetland system than for the shallow wetland or the extended detention shallow wetland systems.
- **Pocket Wetland** – A pocket wetland is intended for smaller drainage areas of 5 to 10 acres and typically requires excavation down to the water table for a reliable water source to support the wetland system.
- **Submerged Gravel -** Also known as subsurface flow wetlands, this wetland consists of one or more cells filled with crushed rock designed to support wetland plants. Storm water flows subsurface through the root zone of the constructed wetland where pollutant removal takes place. *This type of wetland is not recommended for use to meet storm water management goals due to limited performance data.* They may be applicable in special or retrofit situations where there are severe limitations on what can be implemented.

Shallow Wetland

Shallow ED Wetland

Newly Constructed Shallow Wetland

Pocket Wetland

Submerged Gravel Wetland

Figure 5.2.27-1 Storm Water Wetland Examples

5.2.27.2 Storm Water Management Suitability

Similar to storm water ponds, storm water wetlands are designed to control both storm water quantity and quality. Thus, a storm water wetland can be used to address all of the integrated storm water sizing criteria for a given drainage area.

Water Quality

Pollutants are removed from storm water runoff in a wetland through uptake by wetland vegetation and algae, vegetative filtering, and through gravitational settling in the slow moving marsh flow. Other

pollutant removal mechanisms are also at work in a storm water wetland including chemical and biological decomposition and volatilization. Section 5.2.27.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

The storage volume above the permanent pool/water surface level in a storm water wetland is used to provide control of the streambank protection volume (SP_v). This is accomplished by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention). It is best to do this with minimum vertical water level fluctuation, as extreme fluctuation may stress vegetation.

Flood Control

In situations where it is required, storm water wetlands can also be used to provide detention to control the 100-year storm peak flow. Where 100-year peak control is not required, a storm water wetland must be designed to safely pass the 100-year storm flows.

5.2.27.3 Pollutant Removal Capabilities

All of the storm water wetland design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed wetland facilities can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment train” approach.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 40%**
- **Total Nitrogen – 30%**
- **Fecal Coliform – 70% (if no resident waterfowl population present)**
- **Heavy Metals – 50%**

For additional information and data on pollutant removal capabilities for storm water wetlands, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Storm Water Best Management Practices (BMP) Database at www.bmpdatabase.org

Submerged Gravel Wetland

The pollution removal efficiency of the submerged gravel wetland is similar to a typical wetland. Recent data show a TSS removal rate in excess of the 80% goal. This reflects the settling environment of the gravel media. These systems also exhibit removals of about 60% TP, 20% TN, and 50% Zn. The growth of algae and microbes among the gravel media has been determined to be the primary removal mechanism of the submerged gravel wetland.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- **Total Suspended Solids – 80%**
- **Total Phosphorus – 50%**
- **Total Nitrogen – 20%**
- **Fecal Coliform – 70%**

- **Heavy Metals – 50%**

Although gravel wetlands are fairly effective at removing total phosphorus, they have a tendency to contribute small amounts of soluble phosphorus.

5.2.27.4 Application and Site Feasibility Criteria

Storm water wetlands are generally applicable to most types of new development and redevelopment, and can be utilized in both residential and nonresidential areas. However, due to the large land requirements, wetlands may not be practical in higher density areas. The following criteria should be evaluated to ensure the suitability of a storm water wetland for meeting storm water management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – Land requirements may preclude use
- Regional Storm Water Control – YES
- Hot Spot Runoff - YES

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – A minimum of 25 acres and a positive water balance is needed to maintain wetland conditions; 5 acres for pocket wetland
- Space Required – Approximately 3 to 5% of the tributary drainage area
- Site Slope – There should be no more than 8% slope across the wetland site
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 feet; 2 to 3 feet for pocket wetland
- Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer or when treating a hotspot, a separation distance of 2 feet is recommended between the bottom of the wetland and the elevation of the seasonally high water table; pocket wetland is typically below water table.
- Soils – Permeable soils are not well suited for a constructed storm water wetland without a high water table. Underlying soils of hydrologic group "C" or "D" should be adequate to maintain wetland conditions. Most group "A" soils and some group "B" soils will require a liner. *Evaluation of soils should be based upon an actual subsurface analysis and permeability tests*.

5.2.27.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a storm water wetland facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- Storm water wetlands should normally have a minimum contributing drainage area of 25 acres or more. For a pocket wetland, the minimum drainage area is 5 acres.
- A continuous base flow or high water table is required to support wetland vegetation. A water balance must be performed to demonstrate that a storm water wetland can withstand a 30-day drought at summer evaporation rates without completely drawing down (see subsection 2.1.8 for details).
- Wetland siting should also take into account the location and use of other site features such as natural depressions, buffers, and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.

- Storm water wetlands cannot be located within navigable waters of the U.S., including natural wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit. In some isolated cases, a wetlands permit may be granted to convert an existing degraded wetland in the context of local watershed restoration efforts.
- If a wetland facility is not used for flood control less than the 100 year event, it should be designed as an off-line system to bypass higher flows rather than passing them through the wetland system.
- Minimum setback requirements for storm water wetland facilities (when not specified by local ordinance or criteria):
  - From a property line – 10 feet
  - From a private well – 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
  - From a septic system tank/leach field/spray area – 50 feet
- All utilities should be located outside of the wetland site.

B. GENERAL DESIGN

- A well-designed storm water wetland consists of:
  1. Shallow marsh areas of varying depths with wetland vegetation,
  2. Permanent micropool, and
  3. Overlying zone in which runoff control volumes are stored.

Pond/wetland systems also include a storm water pond facility (see Section 5.2.21, *Storm Water Ponds*, for pond design information).

- In addition, all wetland designs must include a sediment forebay at the inflow to the facility to allow heavier sediments to drop out of suspension before the runoff enters the wetland marsh.
- Additional wetland design features include an emergency spillway, maintenance access, safety bench, wetland buffer, and appropriate wetland vegetation and native landscaping.

Figures 5.2.27-3 through 5.2.27-6 in subsection 5.2.27.8 provide plan view and profile schematics for the design of a shallow wetland, extended detention shallow wetland, pond/wetland system, and pocket wetland, respectively.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

In general, wetland designs are unique for each site and application. However, there are a number of geometric ratios and limiting depths for the design of a storm water wetland that must be observed for adequate pollutant removal, ease of maintenance, and improved safety. Table 5.2.27-1 provides the recommended physical specifications and geometry for the various storm water wetland design variants.

Table 5.2.27-1 Recommended Design Criteria for Storm Water Wetlands
Modified from Massachusetts DEP, 1997; Schueler, 1992

Design Criteria	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Length to Width Ratio (minimum)	2:1	2:1	2:1	2:1
Extended Detention (ED)	No	Yes	Optional	Optional
Allocation of WQ_v Volume (pool/marsh/ED) in %	25/75/0	25/25/50	70/30/0 (includes pond volume)	25/75/0
Allocation of Surface Area (deepwater/low marsh/high marsh/semi-wet) in %	20/35/40/5	10/35/45/10	45/25/25/5 (includes pond surface area)	10/45/40/5
Forebay	Required	Required	Required	Required
Micropool	Required	Required	Required	Required
Outlet Configuration	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Hooded broad-crested weir

- The storm water wetland should be designed with the recommended proportion of "depth zones." Each of the four wetland design variants has depth zone allocations which are given as a percentage of the storm water wetland surface area. Target allocations are found in Table 5.2.27-1. The four basic depth zones are:

  Deepwater zone
  From 1.5 to 6 feet deep. Includes the outlet micropool and deepwater channels through the wetland facility. This zone supports little emergent wetland vegetation, but may support submerged or floating vegetation.

  Low marsh zone
  From 6 to 18 inches below the normal permanent pool or water surface elevation. This zone is suitable for the growth of several emergent wetland plant species.

  High marsh zone
  From 6 inches below the pool to the normal pool elevation. This zone will support a greater density and diversity of wetland species than the low marsh zone. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone.

  Semi-wet zone
  Those areas above the permanent pool that are inundated during larger storm events. This zone supports a number of species that can survive flooding.

- A minimum dry weather flow path of 2:1 (length to width) is required from inflow to outlet across the storm water wetland and should ideally be greater than 3:1. This path may be achieved by constructing internal dikes or berms, using marsh plantings, and by using multiple cells. Finger dikes are commonly used in surface flow systems to create serpentine configurations and prevent short-circuiting. Microtopography (contours along the bottom of a wetland or marsh that provide a variety of conditions for different species needs and increases the surface area to volume ratio) is encouraged to enhance wetland diversity.

- A 4- to 6-foot deep micropool must be included in the design at the outlet to prevent the outlet from clogging and resuspension of sediments, and to mitigate thermal effects.

- Maximum depth of any permanent pool areas should generally not exceed 6 feet.
- The volume of the extended detention must not comprise more than 50% of the total WQ_v, and its maximum water surface elevation must not extend more than 3 feet above the normal pool. Q_p and/or SP_v storage can be provided above the maximum WQ_v elevation within the wetland.
- The perimeter of all deep pool areas (4 feet or greater in depth) should be surrounded by safety and aquatic benches similar to those for storm water ponds (see subsection 5.2.21).
- The contours of the wetland should be irregular to provide a more natural landscaping effect.

D. SUBMERGED GRAVEL WETLANDS

- Submerged gravel wetlands should be designed as off-line systems designed to handle only water quality volume.
- Submerged gravel wetland systems need sufficient drainage area to maintain vegetation. See subsection 2.1.12 for guidance on performing a water balance calculation.
- The local slope should be relatively flat (<2%). While there is no minimum slope requirement, there does need to be enough elevation drop from the inlet to the outlet to ensure that hydraulic conveyance by gravity is feasible (generally about 3 to 5 feet).
- A design maximum depth of 16 inches of water at the inlet is recommended, with a total gravel depth of 20 inches.
- Gravel should be 0.5-1.0 inch in size.
- Darcy's Law may be used to estimate flows in the gravel media, although the use of predesign tests with the actual gravel will refine the "effective" hydraulic conductivity.
- The initial design should not utilize more than 70 percent of the potential hydraulic gradient available in the proposed bed to allow a safety factor for clogging.
- Using a value of < 113 $m^3/m^2/d$ for the "effective" hydraulic conductivity (k_s) in the design will also help account for potential clogging.
- An adjustable outlet is recommended to ensure adequate hydraulic gradient and prevent surface flow from occurring and shortcircuiting treatment within the gravel media.
- Washed stone or gravel, should be specified to protect against an accumulation of fine material that could cause hydraulic blockages.
- All submerged gravel wetland designs should include a sediment forebay or other equivalent pretreatment measures to prevent sediment or debris from entering and clogging the gravel bed.
- Unless they receive hotspot runoff, submerged gravel wetland systems can be allowed to intersect the groundwater table.
- Guidance on establishing wetland vegetation can be found in Appendix F, *Landscaping and Aesthetics Guidance*.

E. PRETREATMENT / INLETS

- Sediment regulation is critical to sustain storm water wetlands. A wetland facility should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the storm water flow prior to dispersal into the wetland. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the wetland facility.
- The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The pretreatment storage volume is part of the total WQ_v requirement and may be subtracted from WQ_v for wetland storage sizing.

- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Inflow pipe, channel velocities and exit velocities from the forebay must be nonerosive.

F. OUTLET STRUCTURES

- Flow control from a storm water wetland is typically accomplished with the use of a concrete or corrugated aluminum, aluminized steel, or HDPE riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 5.2.27-2). The riser should be located within the embankment for maintenance access, safety, and aesthetics.
- A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality protection, streambank protection, and flood control runoff volumes. The number of orifices can vary and is usually a function of the pond design.

  For shallow and pocket wetlands, the riser configuration is typically comprised of a streambank protection outlet (usually an orifice) and flood control outlet (often a slot or weir). The streambank protection orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the streambank protection orifice. Thus an off-line shallow or pocket wetland providing *only* water quality treatment can use a simple overflow weir as the outlet structure.

  In the case of a extended detention (ED) shallow wetland, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality extended detention volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the streambank protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

  Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

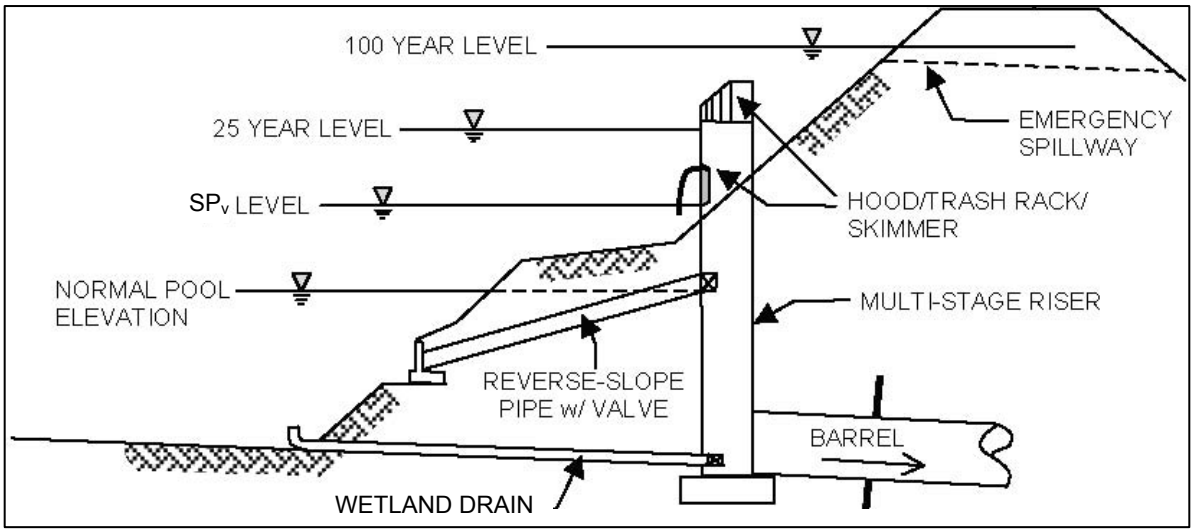


Figure 5.2.27-2 Typical Wetland Facility Outlet Structure

- The water quality outlet (if design is for an extended detention shallow wetland) and streambank protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.
- Higher flows pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the outlet of the barrel to prevent scouring and erosion. If a wetland facility daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Section 4.7 (*Energy Dissipation*) for more guidance.
- The wetland facility must have a bottom drain pipe located in the micropool with an adjustable valve that can completely or partially dewater the wetland within 24 hours.
- The wetland drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

See the design procedures in Section 4.5 (*Storage Design*) and Section 4.6 (*Outlet Structures*) for additional information and specifications on pond routing and outlet works.

G. EMERGENCY SPILLWAY

- An emergency spillway is to be included in the storm water wetland design to safely pass flows that exceed the design storm flows. The spillway prevents the wetland's water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the 100-year flood to the lowest point on top of the dam, not counting the emergency spillway.

H. MAINTENANCE ACCESS

- A maintenance right of way or easement must be provided to the wetland facility from a public road or easement. Maintenance access should be at least 12 feet wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser is to be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.

I. SAFETY FEATURES

- All embankments and spillways must be designed to State of Texas Administrative Code for dams and reservoirs (see Appendix H).
- Fencing of wetlands is not generally desirable, but may be required by the local review authority. A preferred method is to manage the contours of deep pool areas through the inclusion of a safety bench (see above) to eliminate dropoffs and reduce the potential for accidental drowning.
- The principal spillway opening should not permit access by small children, and endwalls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent a hazard.

J. LANDSCAPING

- A landscaping plan should be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include: delineation of landscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed), and sources of plant material.
- Landscaping zones include low marsh, high marsh, and semi-wet zones. The low marsh zone ranges from 6 to 18 inches below the normal pool. This zone is suitable for the growth of several emergent plant species. The high marsh zone ranges from 6 inches below the pool up to the normal pool. This zone will support greater density and diversity of emergent wetland plant species. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone. The semi-wet zone refers to those areas above the permanent pool that are inundated on an irregular basis and can be expected to support wetland plants.
- The landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.
- Woody vegetation may not be planted on a dam embankment or allowed to grow within 15 feet of the toe of the dam and 25 feet from the principal spillway structure.
- A wetland buffer shall extend 25 feet outward from the maximum water surface elevation, with an additional 15-foot setback to structures. The wetland buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall storm water management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident water fowl populations, the buffer can be planted with trees, shrubs and native ground covers.
- The soils of a wetland buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.

Guidance on establishing wetland vegetation can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.

K. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Providing wetland drain can be problematic
- High Relief – Embankment heights restricted
- Karst – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required

Soils

- Hydrologic group "A" soils and some group "B" soils may require liner (not relevant for pocket wetland)

Special Downstream Watershed Considerations

- Local Aquatic Habitat – Design wetland offline and provide shading to reduce thermal impact; limit WQ_v-ED to 12 hours
- Aquifer Protection – Prevent possible groundwater contamination by preventing infiltration of hotspot runoff. May require liner for type "A" soils; Pretreat hotspots; 2 to 4 foot separation distance from water table.

5.2.27.6 Design Procedures

Step 1. Compute runoff control volumes from the *integrated* Design Approach

Calculate the Water Quality Volume (WQ_v), Streambank protection Volume (SP_v), and the 100-year Flood Discharge, (for ED wetlands the design volume should be increased by 15%).

Details on the *integrated* Design Approach are found in Section 1.2.

Step 2. Determine if the development site and conditions are appropriate for the use of a storm water wetland

Consider the Application and Site Feasibility Criteria in subsections 5.2.27.4 and 5.2.27.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 5.2.27.5-K (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The forebay storage volume counts toward the total WQ_v requirement and may be subtracted from the WQ_v for subsequent calculations.

Step 5. Allocate the WQ_v volume among marsh, micropool, and extended detention volumes

Use recommended criteria from Table 5.2.27-1.

Step 6. Determine wetland location and preliminary geometry, including distribution of wetland depth zones

This step involves initially laying out the wetland design and determining the distribution of wetland surface area among the various depth zones (high marsh, low marsh, and deepwater). Set WQ_v permanent pool elevation (and WQ_v-ED elevation for extended detention shallow wetland) based on volumes calculated earlier.

See subsection 5.2.27.5-C (Physical Specification / Geometry) for more details.

Step 7. Compute extended detention orifice release rate(s) and size(s), and establish SP_v elevation

Shallow Wetland and Pocket Wetland: The SP_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool is a recommended design. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

ED Shallow Wetland: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice should have a minimum diameter of 3 inches, and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged one foot below the elevation of the permanent pool, is a recommended design. Adjustable gate valves can also be used to achieve this equivalent diameter. The SP_v elevation is then determined from the stage-storage relationship. The invert of the streambank protection orifice is located at the water quality extended detention elevation, and the orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

Step 8. Calculate the intermediate flood control release rate and water surface elevation

Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the flood control storm.

Step 9. Design embankment(s) and spillway(s)

Size emergency spillway, calculate 100-year water surface elevation, set top of embankment elevation, and analyze safe passage of the 100-year event (Q_f).

At final design, provide safe passage for the 100-year event. Attenuation may not be required.

Step 10. Investigate potential pond/wetland hazard classification

The design and construction of storm water management ponds and wetlands are required to follow the latest version of the State of Texas Administrative Code for dams and reservoirs (see Appendix H).

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See subsection 5.2.27.5-E through I for more details.

Step 12. Prepare Vegetation and Landscaping Plan

A landscaping plan for the wetland facility and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See subsection 5.2.27.5-J (Landscaping) and Appendix F for more details.

5.2.27.7 Inspection and Maintenance Requirements

Table 5.2.27-2 Typical Maintenance Activities for Wetlands
(Adapted from WMI, 1997 and CWP, 1998)

Constructed Wetland Systems	
Activity	**Schedule**
• Replace wetland vegetation to maintain at least 50% surface area coverage in wetland plants after the second growing season.	One-Time Activity
• Clean and remove debris from inlet and outlet structures. • Mow side slopes.	Frequently (3 to 4 times/year)
• Monitor wetland vegetation and perform replacement planting as necessary.	Semi-annual Inspection (first 3 years)
• Examine stability of the original depth zones and microtopographical features. • Inspect for invasive vegetation, and remove where possible. • Inspect for damage to the dam and inlet/outlet structures. Repair as necessary. • Note signs of hydrocarbon build-up, and remove appropriately • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational.	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Harvest wetland plants that have been “choked out” by sediment build-up.	Annually
• Removal of sediment from the forebay	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly, plants are “choked” with sediment, or the wetland becomes eutrophic.	10 to 20 years or after 25% of the wetland volume has been lost
• Ensure that inlets and outlets to each submerged gravel wetland cell are free from debris and not clogged. • Check for sediment buildup in gravel bed.	Monthly Annual inspection
• If sediment buildup is preventing flow through the wetland, remove gravel and sediment from cell. Replace with clean gravel and replant vegetation.	As needed
• Although there is less evaporation with a submerged as opposed to a surface wetland, supplemental water may be required during long periods without storm water input.	As needed
• Routine maintenance of the vegetation (including harvesting) is not required, although weeds can be controlled by flooding the surface after planting and during early part of growing season.	As needed

Additional Maintenance Considerations and Requirements

- Maintenance requirements for constructed wetlands are particularly high while vegetation is being established. Monitoring during these first years is crucial to the future success of the wetland as a storm water structural control. Wetland facilities should be inspected after major storms (greater than 2 inches of rainfall) during the first year of establishment to assess bank stability, erosion damage, flow channelization, and sediment accumulation within the wetland. For the first 3 years, inspections should be conducted at least twice a year.
- A sediment marker should be located in the forebay to determine when sediment removal is required.
- Accumulated sediments will gradually decrease wetland storage and performance. The effects of sediment deposition can be mitigated by the removal of the sediments.
- Sediments excavated from storm water wetlands that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present. Sediment removed from storm water wetlands should be disposed of according to an approved erosion and sediment control plan.
- Periodic mowing of the wetland buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.

Regular inspection and maintenance is critical to the effective operation of storm water wetlands as designed. Maintenance responsibility for a wetland facility and its buffer should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

5.2.27.8 Example Schematics

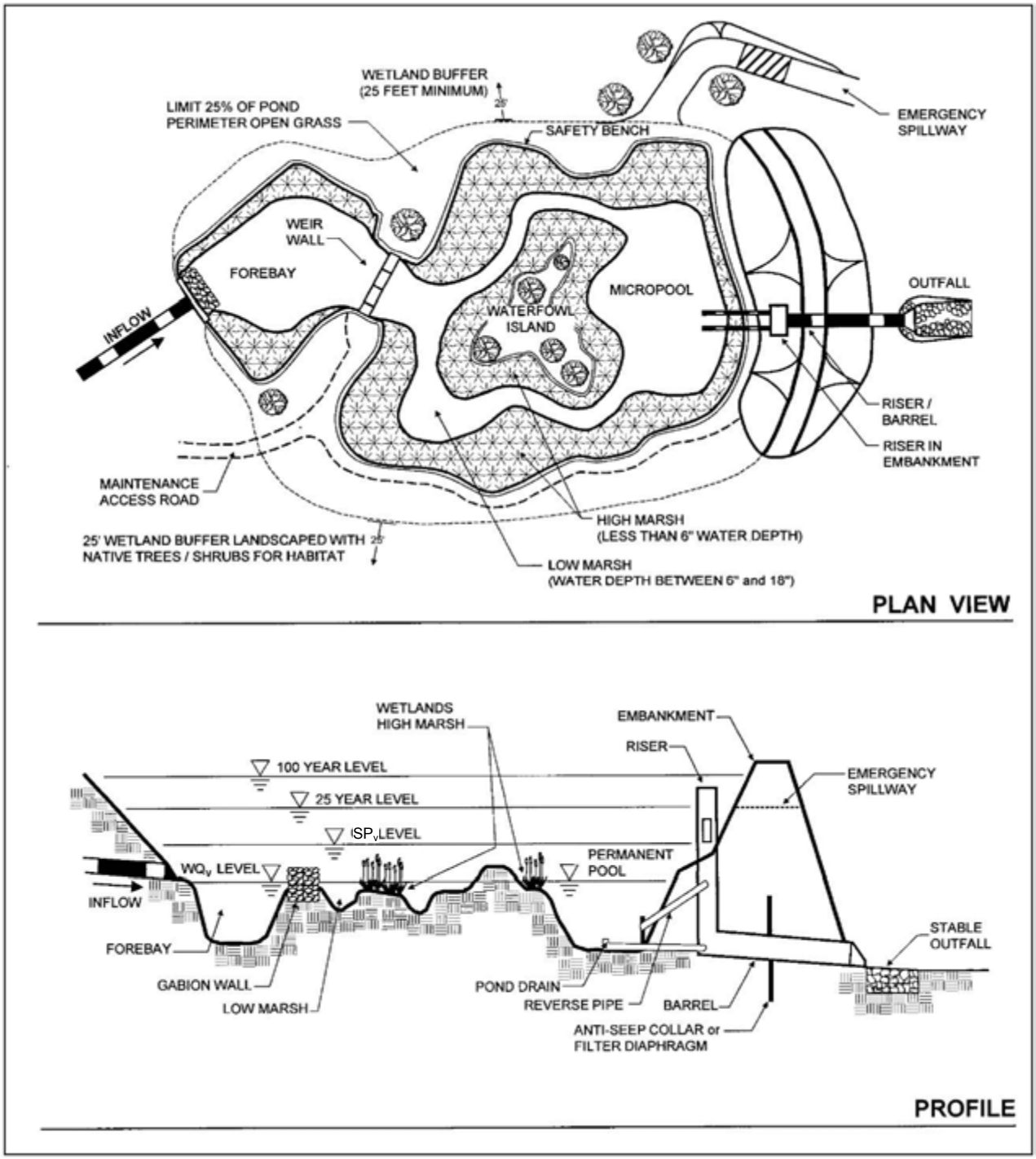


Figure 5.2.27-3 Schematic of Shallow Wetland
(Source: Center for Watershed Protection)

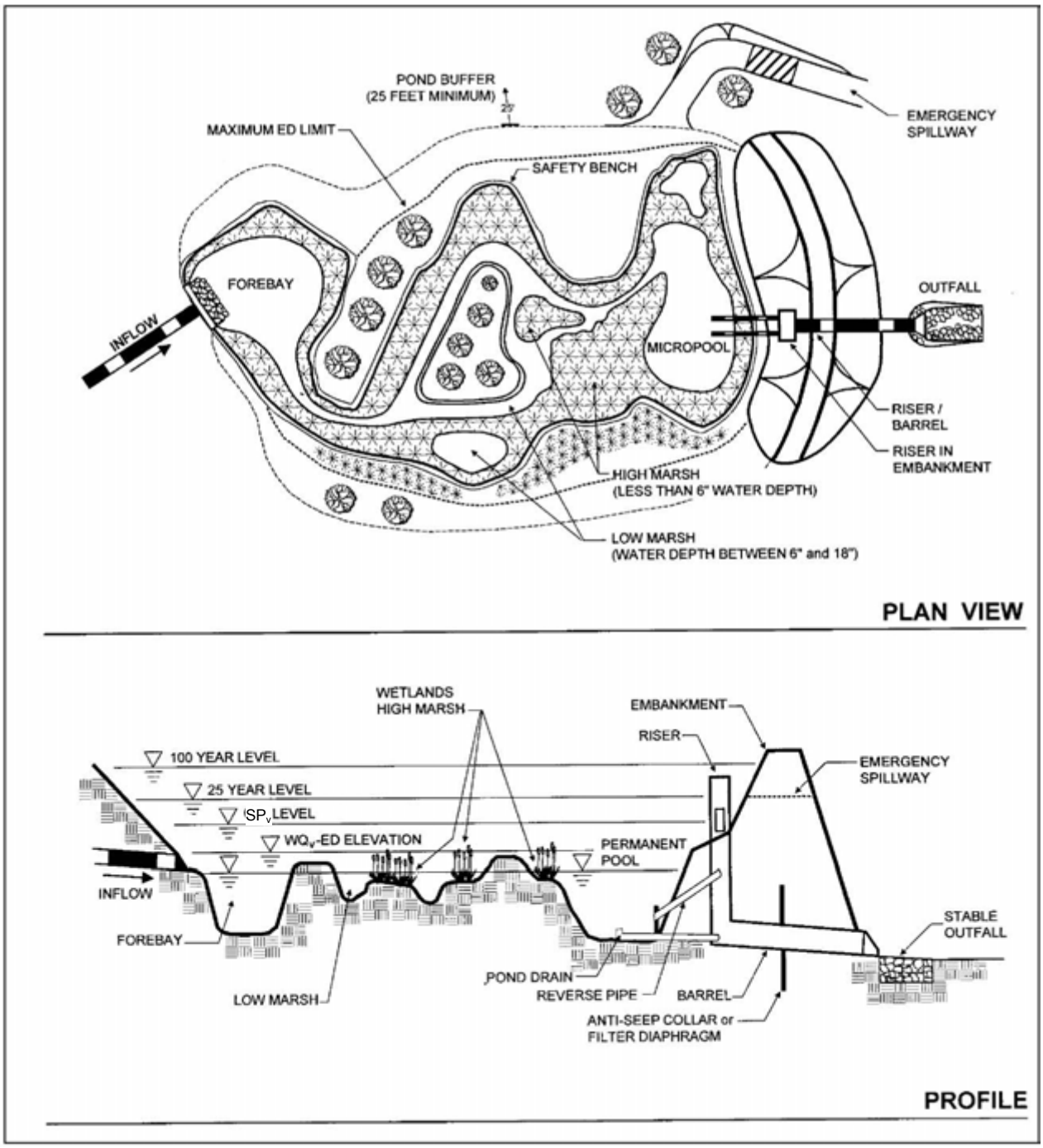


Figure 5.2.27-4 Schematic of Extended Detention Shallow Wetland
(Source: Center for Watershed Protection)

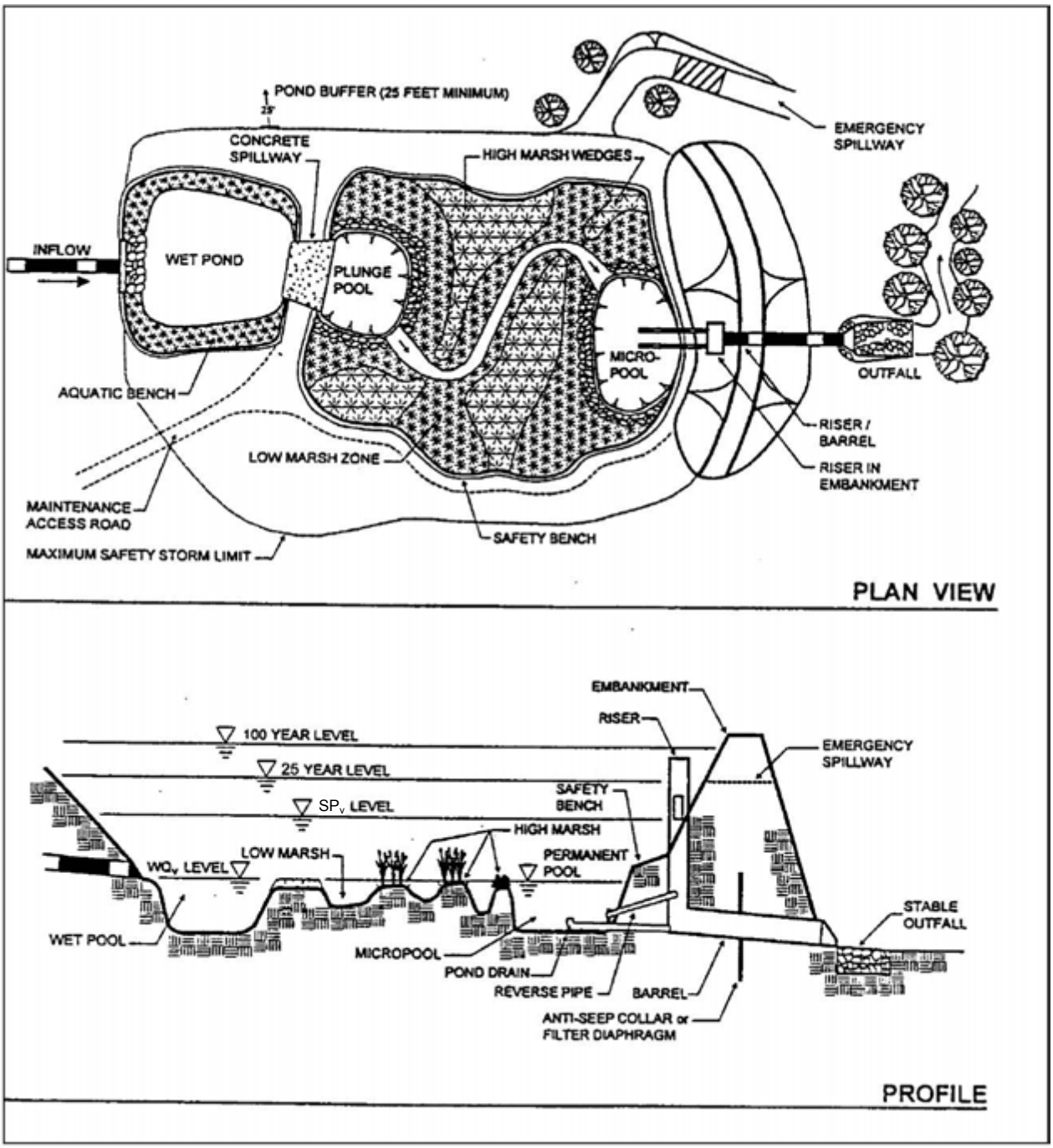


Figure 5.2.27-5 Schematic of Pond/Wetland System
(Source: Center for Watershed Protection)

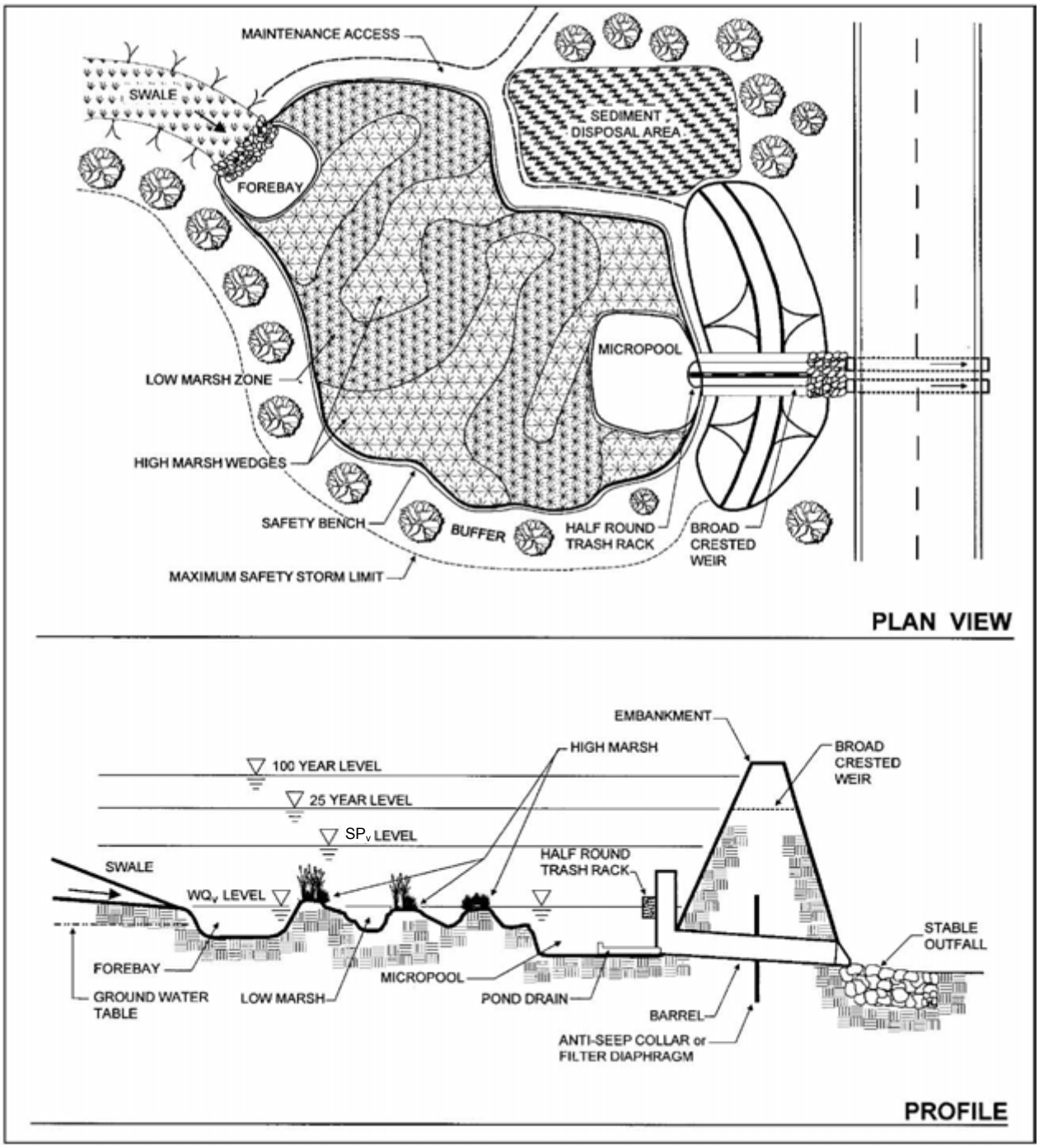
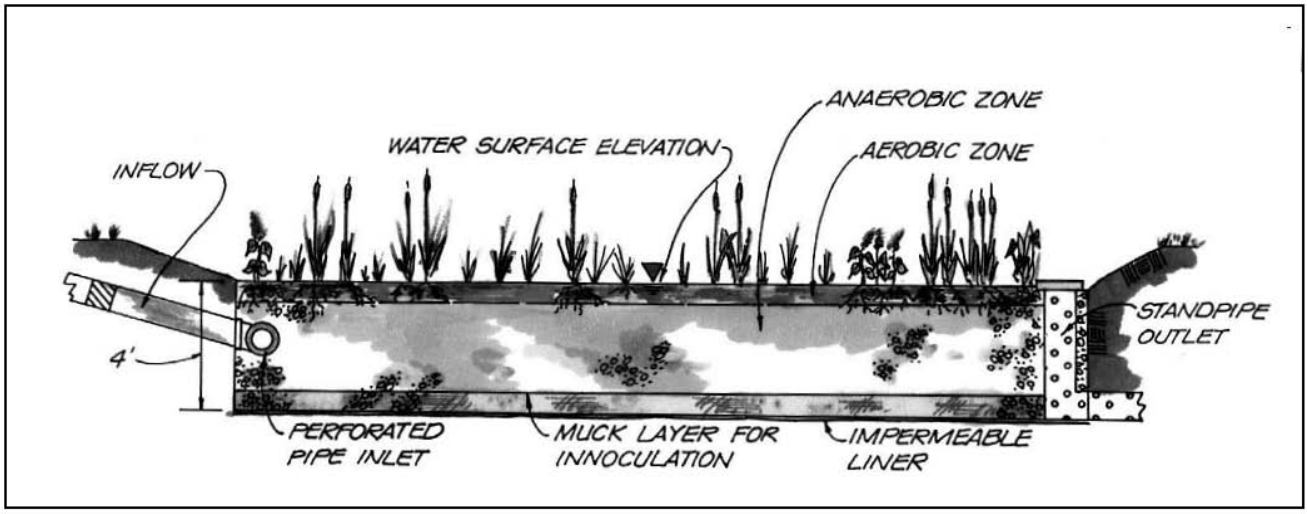


Figure 5.2.27-6 Schematic of Pocket Wetland
(Source: Center for Watershed Protection)



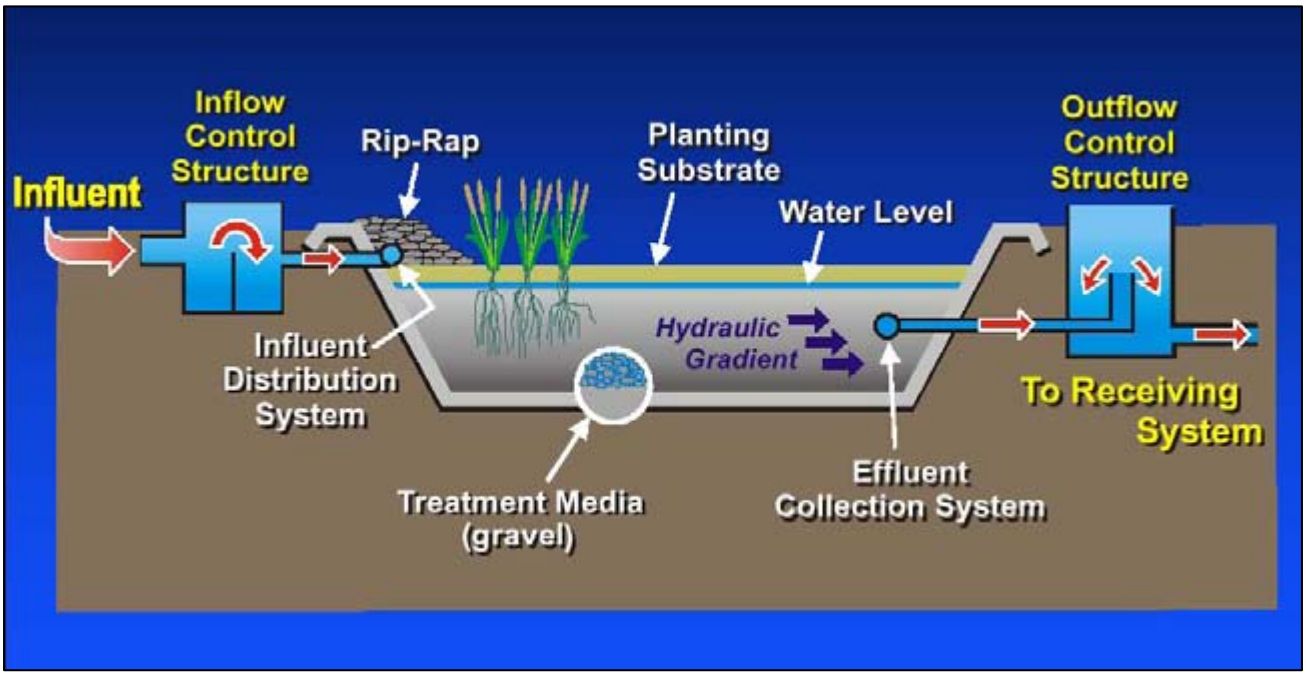


Figure 5.2.27-7 Schematics of Submerged Gravel Wetland System
(Sources: Center for Watershed Protection; Roux Associates Inc.)

5.2.27.9 Design Forms

Design Procedure Form: Storm Water Wetlands

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements
Compute Runoff Coefficient, R_v
Compute WQ_v

R_v = _____
WQ_v = _____ acre-ft

1b. Compute SP_v
Compute average release rate
Compute Q_p (100-year detention volume required)
Add 15% to the required Q_p volume (if ED)
Compute (as necessary) Q_f

SP_v = _____ acre-ft
release rate = _____ cfs
Q_p = _____ acre-ft
Q_p * 15% = _____ acre-ft
Q_f = _____ cfs

STORM WATER WETLAND DESIGN

2. Is the use of a storm water wetland appropriate?

See subsections 5.2.27.4 and 5.2.27.5-A

3. Confirm local design criteria and applicability.

4. Pretreatment volume
Vol_{pre} = 1 (0.1")(1'/12")

Vol_{pre} = _____ acre-ft

5. Allocation of Pool, Marsh, and ED Volumes

Shallow Wetland: Vol_{pool} = 0.25 (WQ_v)
Vol_{marsh} = 0.75 (WQ_v)

Vol_{pool} = _____ acre-ft
Vol_{marsh} = _____ acre-ft

Shallow ED Wetland: Vol_{pool} = 0.25 (WQ_v)
Vol_{marsh} = 0.25 (WQ_v)
Vol_{ED} = 0.5 (WQ_v)

Vol_{pool} = _____ acre-ft
Vol_{marsh} = _____ acre-ft
Vol_{ED} = _____ acre-ft

Pocket Wetland: Vol_{pool} = 0.25 (WQ_v)
Vol_{marsh} = 0.75 (WQ_v)

Vol_{pool} = _____ acre-ft
Vol_{marsh} = _____ acre-ft

6. Allocation of Surface Area
(choose from Table 5.2.27-1 based on wetland variant)
Pool/Deepwater Wetland Zone (1.5-6 feet deep)
Low Marsh Wetland Zone (6-18 inches deep)
High Marsh Wetland Zone (0-6 inches deep)
Semi-Wet Wetland Zone (above pool depth)

$Area_{water}$ = _____ acres, % = _____
$Area_{low}$ = _____ acres, % = _____
$Area_{high}$ = _____ acres, % = _____
$Area_{semi}$ = _____ acres, % = _____

Conduct grading and determine storage available for marsh zones (and ED if applicable), and compute orifice size

Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cummulative Volume ft^3	Cummulative Volume ac-ft	Volume above Permanent Pool ac-ft

Notes: _____

7. WQ_v Orifice Computations
   Average ED release rate (if applicable)
   Average head, h = (ED elev. - Permanent pool elev.)/2
   Area of orifice from orifice equation
   $Q = CA(2gh)^{0.5}$

   Discharge equation $Q = (h)^{0.5}$

   Compute release rate for SP_v-ED control and establish SP_v elevation
   Release rate =
   Average head, h = (SP_v elev. - Permanent pool elev.)/2
   Area of orifice from orifice equation
   $Q = CA(2gh)^{0.5}$
   Discharge equation $Q = (h)^{0.5}$

8. Calculate Q_p release rate and WSEL

release rate _____ cfs
h = _____ ft

A = _____ ft^2
diameter = _____ in
factor = _____ $(h)^{0.5}$

WSEL = _____ ft-NGVD
release rate = _____ cfs
h = _____ ft
A = _____ ft^2
diameter = _____ in
factor = _____ $(h)^{0.5}$

Set up a stage-storage-discharge relationship

Elevation	Storage	Low Flow	Riser			Barrier		Emergency	Total
		WQ_v-ED	SPv-ED	High Storage		Inlet	Pipe	Spillway	Storage
MSL	ac-ft	Ft(ft) Q(cfs)	Ft(ft) Q(cfs)	Orif. Ft Q	Weir Ft Q	Ft(ft) Q(cfs)	Ft(ft) Q(cfs)	Ft(ft) Q(cfs)	Q(cfs)

Q_p = pre-dev. peak discharge - (WQ_v-ED release + SP_v-ED release)

Maximum head =
Use weir equation for slot length ($Q = CLH^{3/2}$)

Check inlet condition
Check outlet condition

Q_p = _____ cfs

H = _____ ft
L = _____ ft

Use culvert charts (Section 4.2)

9. Size emergency spillway, calculate 100-year WSEL and set top of embankment elevation

$WSEL_{25}$ = _____ ft
$WSEL_{100}$ = _____ ft
Q_{ES} = _____ cfs
Q_{ps} = _____ cfs

10. Investigate potential pond hazard classification

See Appendix H

11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.

See subsection 5.2.27.5-D through H

12. Attach landscaping plan (including wetland vegetatoin)

See Appendix F

Notes:

References

California Storm Water Quality Task Force, 1993. California Storm Water Best Management Practice Handbooks.

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Austin, TX, 1988. Water Quality Management. Environmental Criteria Manual. Environmental and Conservation Services.

City of Sacramento, CA, 2000. Guidance Manual for On-Site Stormwater Quality Control Measures. Department of Utilities.

Debo, Thomas N., and Reese, Andrew J., 2003. Municipal Stormwater Management, 2nd Ed. Lewis Publishers, Boca Raton, FL. (See Chapter 13.)

Denver Urban Drainage and Flood Control District (UDFCD), 1999. Urban Storm Drainage Criteria Manual.

Denver Urban Drainage and Flood Control District (UDFCD), 1999. Urban Storm Drainage Manual. Volume 3.

Galli, J., 1990. Peat-Sand Filters: A Proposed Stormwater Management Practice for Urbanized Areas. Metropolitan Washington Council of Governments.

Galli, J., 1992. Analysis of Urban BMP Performance and Longevity in Prince George's County, Maryland. Metropolitan Washington Council of Governments.

Harper, H., and J. Herr. Alum Treatment of Stormwater Runoff: The First Ten Years. Environmental Research & Design, Inc.

Harper, H., J. Herr, and E. Livingston. Alum Treatment of Stormwater Runoff – An Innovative BMP for Urban Runoff Problems. Environmental Research & Design, Inc. Orlando, FL. With Florida Department of Environmental Protection.

LACDPW, 2000 - Los Angeles County Department of Public Works.

Lager, J., W. Smith, R. Finn, and E. Finnemore, 1997. Urban Stormwater Management and Technology: Update and Users Guide. US EPA. EPA-600/8-77-014.

Maryland, 1984. State of Maryland BMP Design Manual, 1984.

Maryland Department of the Environment, 2000. Maryland Stormwater Design Manual, Volumes I and II. Prepared by Center for Watershed Protection (CWP).

Massachusetts Department of Environmental Protection / Massachusetts Office of Coastal Zone Management, 1997. Stormwater Management – Volume One: Stormwater Policy Handbook, and Vlume Two: Stormwater Technical Handbook.

National Pollutant Removal Performance Database, 2nd Ed. *www.cwp.org*

National Storm Water Best Management Practices (BMP) Database. *www.bmpdatabase.org*

Northern Virginia Regional Commission (NVRC), 1992. The Northern Virginia BMP Handbook. Annandale, VA.

Northern Virginia Regional Commission (NVRC), 1992. Underground Detention Tanks as a Best Management Practice.

Pain, Jack, 1992. Portland Cement Pervious Pavement Construction: Vol. 37 No. 9, 4pp.

Pitt, R., M. Lilbum, S. Nix, S. Durrans, and S. Burian, 1997. Guidance Manual for Integrated Wet Weather Flow Collection and Treatment Systems for Newly Urbanized Areas. US EPA. Office of Research and Development.

Rouge River National Wet Weather Demonstration Project (Rouge Project), 1992. *www.wcdoe.ort/rougeriver/*

Schueler, T., 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. Metropolitan Washington Council of Governments.

Soil Conservation Service (SCS), 1986. Urban Hydrology for Small Watersheds. United States Department of Agriculture. Engineering Division. Technical Release 55.

Texas Commission of Environmental Quality. Texas Administrative Code, 30 TAC Part 1, Chapter 299, Dams and Reservoirs. *www.tnrcc.state.tx.us/enforcement/dam_safety/intro2.html*

Texas Natural Resource Conservation Commission, 2000. Texas Nonpoint Source Book. *www.txnpsbook.org*

US EPA, 1999. Storm Water Technology Fact Sheet: Vegetated Swales. EPA 832-F-99-006. Office of Water

US EPA, 1999. Storm Water Technology Fact Sheet: Sand Filters. EPA 832-F-99-007. Office of Water.

US EPA, 1999. Storm Water Technology Fact Sheet: Bioretention. EPA 832-F-99-012. Office of Water

US EPA, 1999. Storm Water Technology Fact Sheet: Infiltration Trench. EPA 832-F-99-019. Office of Water.

US EPA, 1999. Storm Water Technology Fact Sheet: Porous Pavement. EPA 832-F-99-023. Office of Water.

US EPA, 1999. Storm Water Technology Fact Sheet: Storm Water Wetlands. EPA 832-F-99-025. Office of Water.

US EPA, 1999. Storm Water Technology Fact Sheet: Modular Treatment Systems. EPA 832-F-99-044. Office of Water.

Virginia Department of Conservation and Recreation, 1999. Virginia Stormwater Management Handbook, Volumes I and II.

Washington State Department of Ecology, 2000. Stormwater Management Manual for Western Washington.

Watershed Management Institute (WMI), 1997. Operation, Maintenance, and Management of Stormwater Management Systems. Prepared for US EPA, Office of Water.

Wisconsin Department of Natural Resources / University of Wisconsin Extension, 2000. The Wisconsin Stormwater Manual.

Winer, R., 2000. National Pollutant Removal Performance Database for Stormwater BMPs. Center for Watershed Protection. US EPA, Office of Science and Technology.

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APPENDIX

A

RAINFALL TABLES FOR NORTH CENTRAL TEXAS

Methodology A-2

Figure

Figure A-1 County Rainfall Data Location Map A-4

Tables

Table A-1 Collin County A-5
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Methodology

The US Geological Survey (USGS), in cooperation with the Texas Department of Transportation (TxDOT), recently conducted a study of the depth-duration frequency (DDF) of precipitation for Texas[1]. In the study, the frequency of the annual maximum rainfall data was modeled using two probability distributions. The parameters from these distributions can be used to estimate the DDF for any location in Texas.

The results of the study were applied to the hydrologic models used to update the Flood Insurance Rate Maps of Harris County. The City of Austin also asked the USGS for data to update its IDF data. TxDOT will also be updating its intensity-duration frequency (IDF) data statewide.

Depth-duration frequency relationships were obtained from the USGS for each county in the North Central Texas region. The centroid of each county was used as the geographic location for the computations as shown on Figure A-1. The frequency range in the USGS study is from 2 to 500 years and precipitation duration ranges from 15-minutes to 24 hours. The main objective was to find IDF relationships for each of the counties. The results are presented in tabular form in this Appendix for each county.

The main procedure for developing the IDF relationship is shown below.

1. Convert rainfall depths to intensities for each associated duration.
2. Plot intensity vs. duration for each return period on log-log axes.
3. Follow the Dodson Method[2] to develop the IDF relationship shown below.

   $$I = \frac{b}{(T+d)^e}$$

   In the equation above, *I* represents the rainfall intensity in inches/hr, *T* represents the rainfall duration in minutes, and *b, d,* and *e* are coefficients.
4. Determine the IDF relationship for each county in the North Central Texas region, for each return period.

Since it was necessary for the IDF relationships to represent durations less than 15 minutes, the rainfall depths from Hydro-35[3] were obtained for 5 and 10-minute durations. This provided for a smooth transition between the Hydro-35 data and the USGS data.

It was also necessary to develop IDF relationships for the 1-year return period. The procedure that was followed to determine the DDF relationship for each 1-year curve is listed below.

1. Plot depth vs. return period for each duration. (Each curve stops at the 2-year return period.).
2. Extrapolate each curve to the 1-year return period.
3. Convert the extrapolated depths to intensities.

In step 1, the 5 and 10-minute curves were developed using Hydro-35 data, whereas the USGS data was used to develop curves for durations greater than 10 minutes. The extrapolated intensities were then used to develop the 1-year IDF relationship for each county.

Other Comments

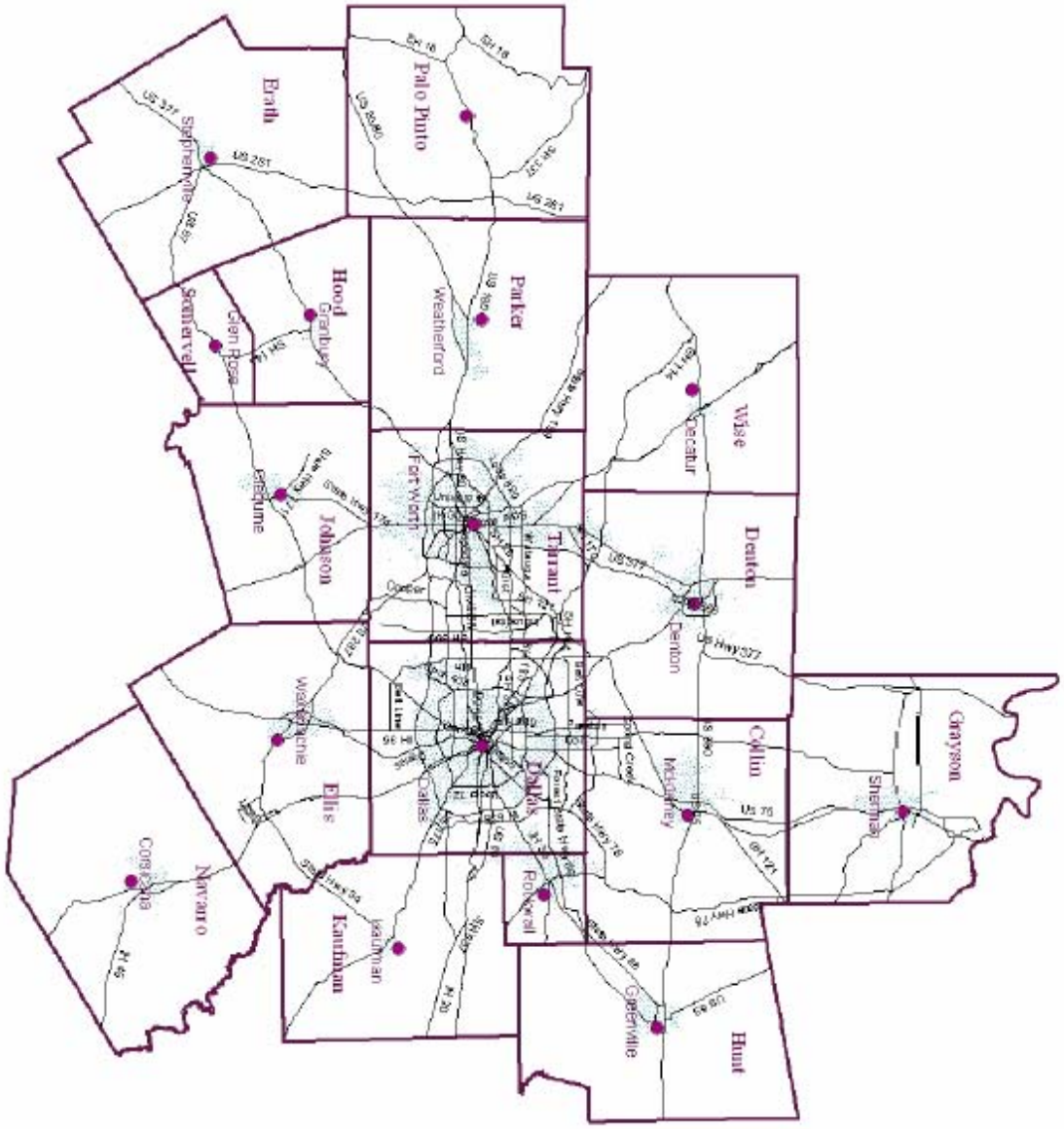
The process of developing IDF relationships involved the merging of DDF data from two different studies. Hydro-35 was published in 1977, but the fitting procedure used for the frequency distribution was developed in 1958. The common practice at the time was to fit the **annual series** to a distribution, then apply correction factors to convert to **partial-duration series**. These empirical factors were used to modify the statistics of the distribution to more closely represent actual precipitation data. In the USGS publication (1998), the **annual series** data was also used, but no conversion was deemed necessary, for the following reasons. First, a greater number of stations and longer periods of records were available at the time of the study. Also, two different distributions were applied for different duration ranges. These distributions were developed more recently (1986, 1990), and make use of L-moments (linear moments), which are more powerful than traditional statistics (mean, standard deviation). As a result of using more advanced statistical analyses, the selected distributions modeled the annual series without the use of conversion factors.

The values reported in the IDF tables are calculated directly from the fitted IDF equations. The fitted equations are only valid for rainfall durations between 5 minutes and 24 hours.

[1]Asquith, W.H. *Depth-Duration Frequency of Precipitation for Texas*. Water-Resources Investigations Report 98-4044, US Geological Survey. Austin, TX, 1998.

[2]Frederick, R.H., V.A. Myers, and E.P. Auciello. NOAA *Technical Memorandum NWS Hydro-35*. Office of Hydrology. Silver Spring, MD, June 1977.

[3]*The Dodson Hydrology Library*. Dodson & Associates, Inc. Houston, TX, 1987.



Legend

- Rainfall Centroids
- Counties
- Major Thoroughfares
- County Seats

North Central Texas Council of Governments
integrated Storm Water Management

County Rainfall Data Location Map

DATE: December 2003

FIGURE A-1

Table A-1 Collin County

		Return Period (Years)						
	Coefficents	1	2	5	10	25	50	100
	e	0.82667	0.79822	0.78901	0.77386	0.75875	0.74805	0.73702
	b	47.053	50.523	64.259	68.951	76.069	81.634	86.709
	d	9	9	11	11	11	11	11
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.31	6.15	7.21	8.07	9.28	10.26	11.24
	6	5.02	5.82	6.87	7.70	8.86	9.80	10.74
	7	4.76	5.52	6.57	7.36	8.49	9.39	10.30
	8	4.52	5.26	6.29	7.06	8.15	9.02	9.90
	9	4.31	5.03	6.05	6.79	7.84	8.68	9.53
	10	4.13	4.82	5.82	6.54	7.55	8.37	9.20
	11	3.95	4.62	5.61	6.31	7.29	8.08	8.89
	12	3.80	4.45	5.41	6.09	7.05	7.82	8.60
	13	3.65	4.28	5.24	5.89	6.82	7.58	8.33
	14	3.52	4.14	5.07	5.71	6.61	7.35	8.09
0.250	15	3.40	4.00	4.91	5.54	6.42	7.14	7.86
	16	3.29	3.87	4.77	5.38	6.24	6.94	7.64
	17	3.18	3.75	4.64	5.23	6.07	6.75	7.44
	18	3.09	3.64	4.51	5.09	5.91	6.58	7.25
	19	2.99	3.53	4.39	4.96	5.76	6.41	7.07
	20	2.91	3.44	4.28	4.84	5.62	6.26	6.90
	21	2.83	3.35	4.17	4.72	5.48	6.11	6.74
	22	2.75	3.26	4.07	4.61	5.36	5.97	6.59
	23	2.68	3.18	3.98	4.50	5.24	5.84	6.45
	24	2.61	3.10	3.89	4.40	5.12	5.71	6.31
	25	2.55	3.03	3.80	4.31	5.02	5.59	6.18
	26	2.49	2.96	3.72	4.22	4.91	5.48	6.06
	27	2.43	2.89	3.64	4.13	4.81	5.37	5.94
	28	2.38	2.83	3.57	4.05	4.72	5.27	5.83
	29	2.33	2.77	3.50	3.97	4.63	5.17	5.72
0.500	30	2.28	2.71	3.43	3.89	4.54	5.07	5.62
	31	2.23	2.66	3.37	3.82	4.46	4.98	5.52
	32	2.18	2.61	3.30	3.75	4.38	4.90	5.42
	33	2.14	2.56	3.25	3.69	4.31	4.81	5.33
	34	2.10	2.51	3.19	3.62	4.23	4.73	5.24
	35	2.06	2.46	3.13	3.56	4.16	4.66	5.16
	36	2.02	2.42	3.08	3.50	4.10	4.58	5.08
	37	1.99	2.38	3.03	3.45	4.03	4.51	5.00
	38	1.95	2.34	2.98	3.39	3.97	4.44	4.92
	39	1.92	2.30	2.93	3.34	3.91	4.37	4.85
	40	1.89	2.26	2.89	3.29	3.85	4.31	4.78
	41	1.85	2.22	2.84	3.24	3.79	4.25	4.71
	42	1.82	2.19	2.80	3.19	3.74	4.19	4.65
	43	1.79	2.16	2.76	3.15	3.69	4.13	4.58
	44	1.77	2.12	2.72	3.10	3.64	4.07	4.52
0.750	45	1.74	2.09	2.68	3.06	3.59	4.02	4.46
	46	1.71	2.06	2.65	3.02	3.54	3.97	4.40
	47	1.69	2.03	2.61	2.98	3.49	3.92	4.35
	48	1.66	2.00	2.57	2.94	3.45	3.87	4.29
	49	1.64	1.98	2.54	2.90	3.40	3.82	4.24
	50	1.62	1.95	2.51	2.86	3.36	3.77	4.19
	51	1.59	1.92	2.48	2.83	3.32	3.72	4.14
	52	1.57	1.90	2.44	2.79	3.28	3.68	4.09
	53	1.55	1.87	2.41	2.76	3.24	3.64	4.04
	54	1.53	1.85	2.39	2.73	3.20	3.60	4.00
	55	1.51	1.83	2.36	2.69	3.17	3.55	3.95
	56	1.49	1.80	2.33	2.66	3.13	3.51	3.91
	57	1.47	1.78	2.30	2.63	3.10	3.48	3.87
	58	1.46	1.76	2.28	2.60	3.06	3.44	3.83
	59	1.44	1.74	2.25	2.57	3.03	3.40	3.79
1	60	1.42	1.72	2.22	2.55	3.00	3.37	3.75
2	120	0.85	1.04	1.37	1.59	1.88	2.13	2.39
3	180	0.62	0.77	1.02	1.18	1.41	1.61	1.81
6	360	0.36	0.45	0.60	0.71	0.85	0.98	1.11
12	720	0.20	0.26	0.35	0.42	0.51	0.59	0.67
24	1440	0.11	0.15	0.21	0.25	0.30	0.35	0.41

Table A-2 Dallas County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.83258	0.81545	0.80449	0.79827	0.78187	0.77019	0.75870
	b	47.679	55.179	70.024	79.931	87.970	94.058	100.079
	d	9	10	12	13	13	13	13
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.30	6.06	7.17	7.96	9.18	10.15	11.17
	6	5.00	5.75	6.85	7.62	8.80	9.74	10.72
	7	4.74	5.48	6.55	7.31	8.45	9.36	10.31
	8	4.51	5.23	6.29	7.03	8.14	9.02	9.94
	9	4.30	5.00	6.05	6.78	7.85	8.70	9.59
	10	4.11	4.80	5.82	6.54	7.58	8.41	9.27
	11	3.94	4.61	5.62	6.32	7.33	8.14	8.98
	12	3.78	4.44	5.43	6.12	7.10	7.88	8.70
	13	3.64	4.28	5.26	5.93	6.89	7.65	8.45
	14	3.50	4.13	5.09	5.76	6.69	7.43	8.21
0.250	15	3.38	4.00	4.94	5.59	6.50	7.22	7.99
	16	3.27	3.87	4.80	5.44	6.32	7.03	7.78
	17	3.16	3.75	4.66	5.29	6.16	6.85	7.58
	18	3.07	3.64	4.54	5.15	6.00	6.68	7.39
	19	2.97	3.54	4.42	5.03	5.85	6.52	7.22
	20	2.89	3.45	4.31	4.90	5.72	6.37	7.05
	21	2.81	3.35	4.20	4.79	5.58	6.22	6.89
	22	2.73	3.27	4.10	4.68	5.46	6.08	6.74
	23	2.66	3.19	4.01	4.57	5.34	5.95	6.60
	24	2.59	3.11	3.92	4.48	5.23	5.83	6.46
	25	2.53	3.04	3.83	4.38	5.12	5.71	6.34
	26	2.47	2.97	3.75	4.29	5.02	5.60	6.21
	27	2.41	2.90	3.68	4.21	4.92	5.49	6.09
	28	2.36	2.84	3.60	4.12	4.82	5.39	5.98
	29	2.31	2.78	3.53	4.05	4.73	5.29	5.87
0.500	30	2.26	2.73	3.46	3.97	4.65	5.19	5.77
	31	2.21	2.67	3.40	3.90	4.56	5.10	5.67
	32	2.17	2.62	3.34	3.83	4.48	5.01	5.57
	33	2.12	2.57	3.28	3.76	4.41	4.93	5.48
	34	2.08	2.52	3.22	3.70	4.33	4.85	5.39
	35	2.04	2.48	3.16	3.64	4.26	4.77	5.31
	36	2.00	2.43	3.11	3.58	4.20	4.69	5.22
	37	1.97	2.39	3.06	3.52	4.13	4.62	5.14
	38	1.93	2.35	3.01	3.46	4.07	4.55	5.07
	39	1.90	2.31	2.96	3.41	4.01	4.48	4.99
	40	1.87	2.27	2.92	3.36	3.95	4.42	4.92
	41	1.84	2.24	2.87	3.31	3.89	4.36	4.85
	42	1.81	2.20	2.83	3.26	3.83	4.30	4.79
	43	1.78	2.17	2.79	3.22	3.78	4.24	4.72
	44	1.75	2.13	2.75	3.17	3.73	4.18	4.66
0.750	45	1.72	2.10	2.71	3.13	3.68	4.12	4.60
	46	1.70	2.07	2.67	3.08	3.63	4.07	4.54
	47	1.67	2.04	2.63	3.04	3.58	4.02	4.48
	48	1.65	2.01	2.60	3.00	3.54	3.97	4.42
	49	1.62	1.98	2.56	2.96	3.49	3.92	4.37
	50	1.60	1.96	2.53	2.93	3.45	3.87	4.32
	51	1.58	1.93	2.50	2.89	3.41	3.82	4.27
	52	1.56	1.91	2.47	2.85	3.36	3.78	4.22
	53	1.53	1.88	2.44	2.82	3.32	3.73	4.17
	54	1.51	1.86	2.41	2.79	3.29	3.69	4.12
	55	1.49	1.83	2.38	2.75	3.25	3.65	4.07
	56	1.48	1.81	2.35	2.72	3.21	3.61	4.03
	57	1.46	1.79	2.32	2.69	3.17	3.57	3.99
	58	1.44	1.77	2.30	2.66	3.14	3.53	3.94
	59	1.42	1.75	2.27	2.63	3.11	3.49	3.90
1	60	1.40	1.73	2.24	2.60	3.07	3.45	3.86
2	120	0.83	1.04	1.38	1.61	1.92	2.18	2.45
3	180	0.61	0.76	1.02	1.20	1.44	1.63	1.85
6	360	0.35	0.44	0.60	0.71	0.86	0.98	1.12
12	720	0.20	0.26	0.35	0.41	0.51	0.58	0.67
24	1440	0.11	0.15	0.20	0.24	0.30	0.35	0.40

Table A-3 Denton County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.82089	0.80553	0.79891	0.78388	0.76912	0.76817	0.75660
	b	43.381	50.455	65.467	70.683	78.538	89.853	95.776
	d	8	9	11	11	11	12	12
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.28	6.02	7.15	8.04	9.31	10.19	11.23
	6	4.97	5.70	6.81	7.67	8.89	9.76	10.75
	7	4.70	5.41	6.50	7.33	8.50	9.36	10.32
	8	4.46	5.15	6.23	7.03	8.16	9.00	9.93
	9	4.24	4.92	5.98	6.75	7.84	8.67	9.57
	10	4.04	4.71	5.75	6.50	7.55	8.36	9.24
	11	3.87	4.52	5.54	6.27	7.29	8.08	8.93
	12	3.71	4.34	5.35	6.05	7.04	7.82	8.65
	13	3.56	4.18	5.17	5.85	6.82	7.58	8.39
	14	3.43	4.04	5.00	5.67	6.61	7.36	8.14
0.250	15	3.31	3.90	4.85	5.50	6.41	7.14	7.91
	16	3.19	3.77	4.70	5.34	6.23	6.95	7.70
	17	3.09	3.66	4.57	5.19	6.05	6.76	7.50
	18	2.99	3.55	4.44	5.05	5.89	6.59	7.31
	19	2.90	3.44	4.32	4.91	5.74	6.43	7.13
	20	2.81	3.35	4.21	4.79	5.60	6.27	6.96
	21	2.73	3.26	4.11	4.67	5.46	6.12	6.80
	22	2.66	3.17	4.01	4.56	5.34	5.99	6.65
	23	2.59	3.09	3.91	4.45	5.21	5.85	6.50
	24	2.52	3.02	3.82	4.35	5.10	5.73	6.36
	25	2.46	2.95	3.74	4.26	4.99	5.61	6.23
	26	2.40	2.88	3.66	4.17	4.89	5.50	6.11
	27	2.34	2.81	3.58	4.08	4.79	5.39	5.99
	28	2.29	2.75	3.51	4.00	4.69	5.28	5.88
	29	2.24	2.69	3.44	3.92	4.60	5.18	5.77
0.500	30	2.19	2.64	3.37	3.85	4.51	5.09	5.66
	31	2.14	2.58	3.31	3.77	4.43	5.00	5.56
	32	2.10	2.53	3.24	3.71	4.35	4.91	5.47
	33	2.06	2.49	3.18	3.64	4.28	4.83	5.38
	34	2.02	2.44	3.13	3.58	4.20	4.75	5.29
	35	1.98	2.39	3.07	3.51	4.13	4.67	5.20
	36	1.94	2.35	3.02	3.46	4.06	4.59	5.12
	37	1.91	2.31	2.97	3.40	4.00	4.52	5.04
	38	1.87	2.27	2.92	3.35	3.94	4.45	4.96
	39	1.84	2.23	2.88	3.29	3.88	4.38	4.89
	40	1.81	2.19	2.83	3.24	3.82	4.32	4.82
	41	1.78	2.16	2.79	3.19	3.76	4.26	4.75
	42	1.75	2.13	2.74	3.15	3.71	4.20	4.68
	43	1.72	2.09	2.70	3.10	3.65	4.14	4.62
	44	1.69	2.06	2.66	3.06	3.60	4.08	4.56
0.750	45	1.67	2.03	2.63	3.01	3.55	4.02	4.50
	46	1.64	2.00	2.59	2.97	3.50	3.97	4.44
	47	1.62	1.97	2.55	2.93	3.46	3.92	4.38
	48	1.59	1.94	2.52	2.89	3.41	3.87	4.32
	49	1.57	1.92	2.49	2.85	3.37	3.82	4.27
	50	1.55	1.89	2.45	2.82	3.33	3.77	4.22
	51	1.53	1.86	2.42	2.78	3.28	3.73	4.17
	52	1.51	1.84	2.39	2.75	3.24	3.68	4.12
	53	1.49	1.82	2.36	2.71	3.21	3.64	4.07
	54	1.47	1.79	2.33	2.68	3.17	3.60	4.02
	55	1.45	1.77	2.30	2.65	3.13	3.55	3.98
	56	1.43	1.75	2.28	2.62	3.09	3.51	3.93
	57	1.41	1.73	2.25	2.59	3.06	3.48	3.89
	58	1.39	1.71	2.22	2.56	3.03	3.44	3.85
	59	1.37	1.69	2.20	2.53	2.99	3.40	3.81
1	60	1.36	1.67	2.17	2.50	2.96	3.36	3.77
2	120	0.81	1.01	1.33	1.55	1.85	2.11	2.38
3	180	0.59	0.74	0.99	1.15	1.38	1.58	1.79
6	360	0.34	0.43	0.58	0.68	0.83	0.95	1.09
12	720	0.19	0.25	0.34	0.40	0.49	0.57	0.65
24	1440	0.11	0.14	0.20	0.23	0.29	0.33	0.39

Table A-4 Ellis County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.84002	0.81147	0.80992	0.80183	0.78513	0.78190	0.76959
	b	51.103	54.710	74.075	83.862	92.418	104.449	110.819
	d	10	10	13	14	14	15	15
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.25	6.08	7.13	7.91	9.16	10.04	11.05
	6	4.98	5.77	6.82	7.59	8.80	9.66	10.64
	7	4.73	5.49	6.55	7.30	8.47	9.32	10.27
	8	4.51	5.24	6.29	7.03	8.16	9.00	9.92
	9	4.31	5.02	6.06	6.79	7.88	8.70	9.60
	10	4.13	4.81	5.84	6.56	7.62	8.43	9.31
	11	3.96	4.63	5.65	6.35	7.38	8.18	9.03
	12	3.81	4.45	5.46	6.15	7.16	7.94	8.77
	13	3.67	4.30	5.29	5.97	6.95	7.72	8.53
	14	3.54	4.15	5.13	5.80	6.75	7.51	8.30
0.250	15	3.42	4.01	4.98	5.64	6.57	7.31	8.09
	16	3.31	3.89	4.84	5.48	6.40	7.13	7.89
	17	3.21	3.77	4.71	5.34	6.24	6.95	7.70
	18	3.11	3.66	4.59	5.21	6.08	6.79	7.52
	19	3.02	3.56	4.47	5.08	5.94	6.63	7.35
	20	2.94	3.46	4.36	4.96	5.80	6.48	7.18
	21	2.86	3.37	4.26	4.85	5.67	6.34	7.03
	22	2.78	3.29	4.16	4.74	5.54	6.20	6.88
	23	2.71	3.21	4.07	4.64	5.43	6.08	6.74
	24	2.64	3.13	3.98	4.54	5.31	5.95	6.61
	25	2.58	3.06	3.89	4.44	5.21	5.84	6.48
	26	2.52	2.99	3.81	4.36	5.10	5.73	6.36
	27	2.46	2.92	3.73	4.27	5.01	5.62	6.24
	28	2.41	2.86	3.66	4.19	4.91	5.52	6.13
	29	2.35	2.80	3.59	4.11	4.82	5.42	6.02
0.500	30	2.31	2.74	3.52	4.03	4.74	5.32	5.92
	31	2.26	2.69	3.46	3.96	4.65	5.23	5.82
	32	2.21	2.64	3.39	3.89	4.57	5.15	5.73
	33	2.17	2.59	3.33	3.83	4.50	5.06	5.63
	34	2.13	2.54	3.28	3.76	4.42	4.98	5.54
	35	2.09	2.49	3.22	3.70	4.35	4.90	5.46
	36	2.05	2.45	3.17	3.64	4.28	4.83	5.38
	37	2.01	2.41	3.12	3.58	4.22	4.76	5.30
	38	1.98	2.36	3.07	3.53	4.15	4.68	5.22
	39	1.94	2.33	3.02	3.48	4.09	4.62	5.15
	40	1.91	2.29	2.97	3.42	4.03	4.55	5.07
	41	1.88	2.25	2.93	3.37	3.98	4.49	5.00
	42	1.85	2.22	2.88	3.33	3.92	4.43	4.94
	43	1.82	2.18	2.84	3.28	3.87	4.37	4.87
	44	1.79	2.15	2.80	3.23	3.81	4.31	4.81
0.750	45	1.76	2.12	2.76	3.19	3.76	4.25	4.74
	46	1.74	2.09	2.73	3.15	3.71	4.20	4.68
	47	1.71	2.06	2.69	3.10	3.66	4.14	4.63
	48	1.69	2.03	2.65	3.06	3.62	4.09	4.57
	49	1.66	2.00	2.62	3.03	3.57	4.04	4.51
	50	1.64	1.97	2.58	2.99	3.53	3.99	4.46
	51	1.62	1.95	2.55	2.95	3.49	3.95	4.41
	52	1.60	1.92	2.52	2.91	3.44	3.90	4.36
	53	1.57	1.90	2.49	2.88	3.40	3.86	4.31
	54	1.55	1.87	2.46	2.85	3.37	3.81	4.26
	55	1.53	1.85	2.43	2.81	3.33	3.77	4.21
	56	1.51	1.83	2.40	2.78	3.29	3.73	4.17
	57	1.49	1.80	2.37	2.75	3.25	3.69	4.12
	58	1.48	1.78	2.35	2.72	3.22	3.65	4.08
	59	1.46	1.76	2.32	2.69	3.18	3.61	4.04
1	60	1.44	1.74	2.29	2.66	3.15	3.57	4.00
2	120	0.86	1.05	1.41	1.65	1.98	2.26	2.54
3	180	0.62	0.77	1.04	1.23	1.48	1.69	1.92
6	360	0.36	0.45	0.61	0.73	0.88	1.01	1.16
12	720	0.20	0.26	0.35	0.42	0.52	0.60	0.69
24	1440	0.11	0.15	0.20	0.24	0.30	0.35	0.41

Table A-5 Erath County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.80626	0.81545	0.80871	0.80362	0.79735	0.78645	0.78371
	b	37.942	49.894	65.302	75.466	89.169	96.255	109.034
	d	7	9	11	12	13	13	14
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.12	5.80	6.94	7.74	8.90	9.91	10.85
	6	4.80	5.48	6.60	7.40	8.52	9.50	10.42
	7	4.52	5.20	6.31	7.08	8.18	9.12	10.03
	8	4.27	4.95	6.04	6.80	7.87	8.78	9.67
	9	4.06	4.73	5.79	6.53	7.58	8.47	9.34
	10	3.86	4.52	5.57	6.29	7.32	8.18	9.03
	11	3.69	4.34	5.36	6.07	7.07	7.91	8.75
	12	3.53	4.17	5.17	5.87	6.85	7.66	8.48
	13	3.39	4.01	5.00	5.68	6.64	7.42	8.24
	14	3.26	3.87	4.84	5.50	6.44	7.21	8.01
0.250	15	3.14	3.74	4.68	5.34	6.26	7.00	7.79
	16	3.03	3.61	4.54	5.19	6.08	6.81	7.58
	17	2.93	3.50	4.41	5.04	5.92	6.63	7.39
	18	2.83	3.40	4.29	4.91	5.77	6.46	7.21
	19	2.74	3.30	4.17	4.78	5.62	6.31	7.04
	20	2.66	3.20	4.06	4.66	5.49	6.15	6.88
	21	2.58	3.12	3.96	4.54	5.36	6.01	6.72
	22	2.51	3.03	3.86	4.44	5.24	5.88	6.57
	23	2.44	2.96	3.77	4.33	5.12	5.75	6.43
	24	2.38	2.88	3.68	4.24	5.01	5.62	6.30
	25	2.32	2.81	3.60	4.14	4.90	5.51	6.17
	26	2.26	2.75	3.52	4.06	4.80	5.40	6.05
	27	2.21	2.69	3.45	3.97	4.71	5.29	5.94
	28	2.16	2.63	3.37	3.89	4.62	5.19	5.83
	29	2.11	2.57	3.31	3.82	4.53	5.09	5.72
0.500	30	2.06	2.52	3.24	3.74	4.44	5.00	5.62
	31	2.02	2.46	3.18	3.67	4.36	4.91	5.52
	32	1.98	2.41	3.12	3.61	4.29	4.82	5.43
	33	1.94	2.37	3.06	3.54	4.21	4.74	5.33
	34	1.90	2.32	3.01	3.48	4.14	4.66	5.25
	35	1.86	2.28	2.95	3.42	4.07	4.58	5.16
	36	1.83	2.24	2.90	3.36	4.00	4.51	5.08
	37	1.80	2.20	2.85	3.31	3.94	4.44	5.00
	38	1.76	2.16	2.81	3.25	3.88	4.37	4.93
	39	1.73	2.12	2.76	3.20	3.82	4.30	4.86
	40	1.70	2.09	2.72	3.15	3.76	4.24	4.78
	41	1.67	2.05	2.67	3.11	3.71	4.18	4.72
	42	1.65	2.02	2.63	3.06	3.65	4.12	4.65
	43	1.62	1.99	2.59	3.01	3.60	4.06	4.59
	44	1.59	1.96	2.56	2.97	3.55	4.00	4.52
0.750	45	1.57	1.93	2.52	2.93	3.50	3.95	4.46
	46	1.54	1.90	2.48	2.89	3.45	3.90	4.41
	47	1.52	1.87	2.45	2.85	3.41	3.85	4.35
	48	1.50	1.85	2.41	2.81	3.36	3.80	4.29
	49	1.48	1.82	2.38	2.77	3.32	3.75	4.24
	50	1.46	1.79	2.35	2.74	3.28	3.70	4.19
	51	1.44	1.77	2.32	2.70	3.24	3.66	4.14
	52	1.42	1.75	2.29	2.67	3.20	3.61	4.09
	53	1.40	1.72	2.26	2.64	3.16	3.57	4.04
	54	1.38	1.70	2.23	2.60	3.12	3.53	3.99
	55	1.36	1.68	2.21	2.57	3.08	3.49	3.95
	56	1.34	1.66	2.18	2.54	3.05	3.45	3.90
	57	1.33	1.64	2.15	2.51	3.01	3.41	3.86
	58	1.31	1.62	2.13	2.48	2.98	3.37	3.82
	59	1.29	1.60	2.10	2.45	2.95	3.33	3.78
1	60	1.28	1.58	2.08	2.43	2.91	3.30	3.74
2	120	0.76	0.95	1.27	1.49	1.81	2.06	2.35
3	180	0.56	0.69	0.93	1.10	1.34	1.53	1.76
6	360	0.32	0.40	0.55	0.65	0.79	0.91	1.05
12	720	0.19	0.23	0.32	0.38	0.46	0.54	0.62
24	1440	0.11	0.13	0.18	0.22	0.27	0.31	0.36

Table A-6 Grayson County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.81558	0.79997	0.78004	0.77400	0.75825	0.74672	0.73522
	b	43.546	50.367	59.963	68.347	75.379	80.570	85.722
	d	8	9	10	11	11	11	11
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.38	6.10	7.25	7.99	9.21	10.16	11.16
	6	5.06	5.77	6.90	7.63	8.80	9.71	10.68
	7	4.78	5.48	6.58	7.30	8.42	9.31	10.24
	8	4.54	5.22	6.29	7.00	8.08	8.94	9.84
	9	4.32	4.99	6.03	6.73	7.78	8.60	9.47
	10	4.12	4.78	5.79	6.48	7.49	8.30	9.14
	11	3.94	4.59	5.58	6.25	7.23	8.01	8.83
	12	3.78	4.41	5.38	6.04	6.99	7.75	8.55
	13	3.64	4.25	5.20	5.84	6.77	7.51	8.29
	14	3.50	4.10	5.03	5.66	6.57	7.28	8.04
0.250	15	3.38	3.96	4.87	5.49	6.37	7.07	7.81
	16	3.26	3.84	4.72	5.33	6.19	6.88	7.60
	17	3.15	3.72	4.59	5.18	6.02	6.69	7.40
	18	3.05	3.61	4.46	5.04	5.87	6.52	7.21
	19	2.96	3.50	4.34	4.91	5.72	6.36	7.03
	20	2.88	3.41	4.22	4.79	5.58	6.20	6.86
	21	2.79	3.32	4.12	4.67	5.44	6.06	6.71
	22	2.72	3.23	4.02	4.56	5.32	5.92	6.56
	23	2.65	3.15	3.92	4.46	5.20	5.79	6.41
	24	2.58	3.07	3.83	4.36	5.09	5.66	6.28
	25	2.51	3.00	3.75	4.27	4.98	5.55	6.15
	26	2.45	2.93	3.66	4.18	4.88	5.43	6.03
	27	2.40	2.87	3.59	4.09	4.78	5.33	5.91
	28	2.34	2.80	3.51	4.01	4.69	5.23	5.80
	29	2.29	2.74	3.44	3.93	4.60	5.13	5.69
0.500	30	2.24	2.69	3.37	3.86	4.51	5.03	5.59
	31	2.19	2.63	3.31	3.79	4.43	4.94	5.49
	32	2.15	2.58	3.25	3.72	4.35	4.86	5.40
	33	2.11	2.53	3.19	3.65	4.28	4.78	5.31
	34	2.07	2.49	3.13	3.59	4.20	4.70	5.22
	35	2.03	2.44	3.08	3.53	4.13	4.62	5.14
	36	1.99	2.40	3.03	3.47	4.07	4.55	5.06
	37	1.95	2.35	2.98	3.42	4.00	4.47	4.98
	38	1.92	2.31	2.93	3.36	3.94	4.41	4.90
	39	1.88	2.28	2.88	3.31	3.88	4.34	4.83
	40	1.85	2.24	2.84	3.26	3.82	4.28	4.76
	41	1.82	2.20	2.79	3.21	3.77	4.22	4.69
	42	1.79	2.17	2.75	3.16	3.71	4.16	4.63
	43	1.76	2.13	2.71	3.12	3.66	4.10	4.56
	44	1.74	2.10	2.67	3.07	3.61	4.04	4.50
0.750	45	1.71	2.07	2.63	3.03	3.56	3.99	4.44
	46	1.68	2.04	2.60	2.99	3.51	3.94	4.39
	47	1.66	2.01	2.56	2.95	3.47	3.89	4.33
	48	1.63	1.98	2.53	2.91	3.42	3.84	4.28
	49	1.61	1.96	2.49	2.87	3.38	3.79	4.22
	50	1.59	1.93	2.46	2.84	3.34	3.74	4.17
	51	1.57	1.90	2.43	2.80	3.30	3.70	4.12
	52	1.54	1.88	2.40	2.77	3.26	3.65	4.08
	53	1.52	1.85	2.37	2.73	3.22	3.61	4.03
	54	1.50	1.83	2.34	2.70	3.18	3.57	3.98
	55	1.48	1.81	2.31	2.67	3.14	3.53	3.94
	56	1.47	1.79	2.28	2.64	3.11	3.49	3.90
	57	1.45	1.76	2.26	2.61	3.07	3.45	3.85
	58	1.43	1.74	2.23	2.58	3.04	3.41	3.81
	59	1.41	1.72	2.21	2.55	3.01	3.38	3.77
1	60	1.39	1.70	2.18	2.52	2.98	3.34	3.73
2	120	0.83	1.03	1.35	1.57	1.87	2.11	2.38
3	180	0.61	0.76	1.00	1.17	1.40	1.60	1.80
6	360	0.35	0.45	0.60	0.70	0.85	0.97	1.11
12	720	0.20	0.26	0.35	0.42	0.51	0.59	0.67
24	1440	0.12	0.15	0.21	0.24	0.30	0.35	0.41

Table A-7 Hood County

	Coefficients	Return Period (Years) 1	2	5	10	25	50	100
	e	0.81637	0.81319	0.80743	0.80275	0.79730	0.79586	0.78454
	b	41.477	50.451	66.098	76.415	90.450	103.535	110.783
	d	8	9	11	12	13	14	14
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.11	5.90	7.05	7.86	9.03	9.94	11.00
	6	4.81	5.58	6.71	7.51	8.65	9.54	10.56
	7	4.55	5.29	6.41	7.19	8.30	9.18	10.17
	8	4.31	5.04	6.13	6.90	7.98	8.85	9.80
	9	4.10	4.81	5.88	6.63	7.69	8.54	9.47
	10	3.92	4.60	5.66	6.39	7.43	8.25	9.15
	11	3.75	4.41	5.45	6.17	7.18	7.99	8.87
	12	3.59	4.24	5.26	5.96	6.95	7.74	8.60
	13	3.45	4.09	5.08	5.77	6.73	7.51	8.35
	14	3.33	3.94	4.91	5.59	6.53	7.30	8.11
0.250	15	3.21	3.81	4.76	5.42	6.35	7.10	7.89
	16	3.10	3.68	4.62	5.27	6.17	6.91	7.68
	17	3.00	3.57	4.48	5.12	6.01	6.73	7.49
	18	2.90	3.46	4.36	4.98	5.85	6.56	7.31
	19	2.81	3.36	4.24	4.85	5.71	6.41	7.13
	20	2.73	3.26	4.13	4.73	5.57	6.26	6.97
	21	2.65	3.17	4.03	4.62	5.44	6.11	6.81
	22	2.58	3.09	3.93	4.51	5.31	5.98	6.66
	23	2.51	3.01	3.83	4.40	5.19	5.85	6.52
	24	2.45	2.94	3.75	4.30	5.08	5.73	6.38
	25	2.39	2.87	3.66	4.21	4.98	5.61	6.25
	26	2.33	2.80	3.58	4.12	4.87	5.50	6.13
	27	2.28	2.74	3.50	4.04	4.78	5.39	6.01
	28	2.22	2.68	3.43	3.95	4.68	5.29	5.90
	29	2.18	2.62	3.36	3.88	4.59	5.19	5.79
0.500	30	2.13	2.56	3.30	3.80	4.51	5.09	5.69
	31	2.08	2.51	3.23	3.73	4.43	5.00	5.59
	32	2.04	2.46	3.17	3.66	4.35	4.92	5.50
	33	2.00	2.41	3.11	3.60	4.27	4.83	5.40
	34	1.96	2.37	3.06	3.54	4.20	4.75	5.31
	35	1.92	2.33	3.00	3.47	4.13	4.68	5.23
	36	1.89	2.28	2.95	3.42	4.06	4.60	5.15
	37	1.85	2.24	2.90	3.36	4.00	4.53	5.07
	38	1.82	2.20	2.85	3.31	3.94	4.46	4.99
	39	1.79	2.17	2.81	3.25	3.87	4.39	4.92
	40	1.76	2.13	2.76	3.20	3.82	4.33	4.85
	41	1.73	2.10	2.72	3.16	3.76	4.27	4.78
	42	1.70	2.06	2.68	3.11	3.71	4.21	4.71
	43	1.67	2.03	2.64	3.06	3.65	4.15	4.64
	44	1.65	2.00	2.60	3.02	3.60	4.09	4.58
0.750	45	1.62	1.97	2.56	2.98	3.55	4.03	4.52
	46	1.60	1.94	2.53	2.93	3.50	3.98	4.46
	47	1.57	1.91	2.49	2.89	3.46	3.93	4.40
	48	1.55	1.88	2.46	2.86	3.41	3.88	4.35
	49	1.53	1.86	2.42	2.82	3.37	3.83	4.29
	50	1.51	1.83	2.39	2.78	3.32	3.78	4.24
	51	1.49	1.81	2.36	2.75	3.28	3.73	4.19
	52	1.47	1.78	2.33	2.71	3.24	3.69	4.14
	53	1.45	1.76	2.30	2.68	3.20	3.65	4.09
	54	1.43	1.74	2.27	2.65	3.17	3.60	4.04
	55	1.41	1.71	2.24	2.61	3.13	3.56	4.00
	56	1.39	1.69	2.22	2.58	3.09	3.52	3.95
	57	1.37	1.67	2.19	2.55	3.06	3.48	3.91
	58	1.36	1.65	2.16	2.52	3.02	3.44	3.87
	59	1.34	1.63	2.14	2.50	2.99	3.41	3.82
1	60	1.32	1.61	2.12	2.47	2.96	3.37	3.78
2	120	0.79	0.97	1.29	1.52	1.83	2.10	2.38
3	180	0.58	0.71	0.95	1.12	1.36	1.56	1.78
6	360	0.33	0.41	0.56	0.66	0.81	0.93	1.06
12	720	0.19	0.24	0.32	0.38	0.47	0.54	0.63
24	1440	0.11	0.14	0.19	0.22	0.27	0.31	0.37

Table A-8 Hunt County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.82939	0.80293	0.78081	0.76472	0.74870	0.73779	0.72681
	b	50.510	53.647	62.767	66.895	73.271	78.271	83.114
	d	10	10	11	11	11	11	11
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.34	6.10	7.20	8.03	9.19	10.12	11.08
	6	5.07	5.79	6.87	7.66	8.78	9.68	10.60
	7	4.82	5.52	6.57	7.34	8.42	9.28	10.17
	8	4.59	5.27	6.30	7.04	8.08	8.92	9.78
	9	4.39	5.04	6.05	6.77	7.78	8.58	9.42
	10	4.21	4.84	5.83	6.52	7.50	8.28	9.09
	11	4.04	4.65	5.62	6.29	7.24	8.00	8.79
	12	3.89	4.48	5.43	6.08	7.00	7.74	8.51
	13	3.75	4.33	5.25	5.89	6.79	7.50	8.25
	14	3.62	4.18	5.08	5.71	6.58	7.28	8.01
0.250	15	3.50	4.05	4.93	5.54	6.39	7.07	7.79
	16	3.39	3.92	4.79	5.38	6.21	6.88	7.57
	17	3.28	3.80	4.65	5.23	6.05	6.70	7.38
	18	3.19	3.69	4.53	5.09	5.89	6.53	7.19
	19	3.09	3.59	4.41	4.96	5.74	6.36	7.02
	20	3.01	3.50	4.30	4.84	5.60	6.21	6.85
	21	2.93	3.40	4.19	4.72	5.47	6.07	6.69
	22	2.85	3.32	4.09	4.61	5.35	5.93	6.55
	23	2.78	3.24	4.00	4.51	5.23	5.80	6.41
	24	2.71	3.16	3.91	4.41	5.12	5.68	6.27
	25	2.65	3.09	3.82	4.32	5.01	5.56	6.15
	26	2.59	3.02	3.74	4.23	4.91	5.45	6.02
	27	2.53	2.95	3.67	4.14	4.81	5.35	5.91
	28	2.47	2.89	3.59	4.06	4.72	5.24	5.80
	29	2.42	2.83	3.52	3.98	4.63	5.15	5.69
0.500	30	2.37	2.77	3.46	3.91	4.54	5.05	5.59
	31	2.32	2.72	3.39	3.84	4.46	4.97	5.49
	32	2.28	2.67	3.33	3.77	4.38	4.88	5.40
	33	2.23	2.62	3.27	3.70	4.31	4.80	5.31
	34	2.19	2.57	3.21	3.64	4.24	4.72	5.23
	35	2.15	2.52	3.16	3.58	4.17	4.64	5.14
	36	2.11	2.48	3.11	3.52	4.10	4.57	5.06
	37	2.07	2.44	3.05	3.47	4.04	4.50	4.99
	38	2.04	2.40	3.01	3.41	3.98	4.43	4.91
	39	2.00	2.36	2.96	3.36	3.92	4.37	4.84
	40	1.97	2.32	2.91	3.31	3.86	4.30	4.77
	41	1.94	2.28	2.87	3.26	3.80	4.24	4.70
	42	1.91	2.25	2.83	3.21	3.75	4.18	4.64
	43	1.88	2.21	2.79	3.17	3.70	4.13	4.58
	44	1.85	2.18	2.75	3.12	3.65	4.07	4.52
0.750	45	1.82	2.15	2.71	3.08	3.60	4.02	4.46
	46	1.79	2.12	2.67	3.04	3.55	3.96	4.40
	47	1.77	2.09	2.64	3.00	3.50	3.91	4.35
	48	1.74	2.06	2.60	2.96	3.46	3.86	4.29
	49	1.72	2.03	2.57	2.92	3.42	3.82	4.24
	50	1.69	2.00	2.53	2.88	3.37	3.77	4.19
	51	1.67	1.98	2.50	2.85	3.33	3.73	4.14
	52	1.65	1.95	2.47	2.81	3.29	3.68	4.09
	53	1.63	1.93	2.44	2.78	3.26	3.64	4.05
	54	1.60	1.90	2.41	2.75	3.22	3.60	4.00
	55	1.58	1.88	2.38	2.72	3.18	3.56	3.96
	56	1.56	1.86	2.35	2.69	3.15	3.52	3.91
	57	1.54	1.83	2.33	2.65	3.11	3.48	3.87
	58	1.53	1.81	2.30	2.63	3.08	3.44	3.83
	59	1.51	1.79	2.28	2.60	3.04	3.41	3.79
1	60	1.49	1.77	2.25	2.57	3.01	3.37	3.75
2	120	0.89	1.08	1.39	1.61	1.90	2.15	2.40
3	180	0.65	0.79	1.04	1.21	1.44	1.62	1.83
6	360	0.37	0.47	0.62	0.73	0.87	1.00	1.13
12	720	0.21	0.27	0.36	0.43	0.53	0.60	0.69
24	1440	0.12	0.16	0.21	0.26	0.31	0.36	0.42

Table A-9 Johnson County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.81894	0.80951	0.81158	0.80523	0.78945	0.78704	0.78336
	b	42.817	50.706	70.148	80.274	89.082	101.325	113.822
	d	8	9	12	13	13	14	15
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.24	5.99	7.04	7.83	9.10	9.98	10.89
	6	4.93	5.66	6.72	7.50	8.72	9.59	10.48
	7	4.66	5.37	6.43	7.19	8.37	9.23	10.11
	8	4.42	5.12	6.17	6.92	8.05	8.90	9.76
	9	4.21	4.89	5.93	6.66	7.76	8.59	9.44
	10	4.01	4.68	5.71	6.43	7.50	8.31	9.14
	11	3.84	4.49	5.51	6.21	7.25	8.04	8.87
	12	3.68	4.31	5.32	6.01	7.02	7.80	8.61
	13	3.54	4.15	5.15	5.82	6.80	7.57	8.37
	14	3.41	4.01	4.98	5.65	6.60	7.36	8.14
0.250	15	3.28	3.87	4.83	5.49	6.42	7.16	7.93
	16	3.17	3.74	4.69	5.33	6.24	6.97	7.73
	17	3.07	3.63	4.56	5.19	6.08	6.79	7.54
	18	2.97	3.52	4.44	5.05	5.92	6.62	7.36
	19	2.88	3.42	4.32	4.93	5.77	6.47	7.19
	20	2.80	3.32	4.21	4.81	5.64	6.32	7.03
	21	2.72	3.23	4.11	4.69	5.51	6.17	6.87
	22	2.64	3.15	4.01	4.58	5.38	6.04	6.73
	23	2.57	3.07	3.92	4.48	5.26	5.91	6.59
	24	2.51	2.99	3.83	4.38	5.15	5.79	6.45
	25	2.44	2.92	3.74	4.29	5.04	5.67	6.33
	26	2.38	2.85	3.66	4.20	4.94	5.56	6.21
	27	2.33	2.79	3.59	4.12	4.84	5.45	6.09
	28	2.28	2.73	3.51	4.04	4.75	5.35	5.98
	29	2.23	2.67	3.44	3.96	4.66	5.25	5.87
0.500	30	2.18	2.61	3.38	3.88	4.57	5.16	5.77
	31	2.13	2.56	3.31	3.81	4.49	5.06	5.67
	32	2.09	2.51	3.25	3.74	4.41	4.98	5.58
	33	2.05	2.46	3.19	3.68	4.34	4.89	5.49
	34	2.01	2.41	3.14	3.62	4.26	4.81	5.40
	35	1.97	2.37	3.08	3.55	4.19	4.74	5.31
	36	1.93	2.33	3.03	3.50	4.13	4.66	5.23
	37	1.90	2.29	2.98	3.44	4.06	4.59	5.15
	38	1.86	2.25	2.93	3.39	4.00	4.52	5.08
	39	1.83	2.21	2.89	3.33	3.94	4.45	5.00
	40	1.80	2.17	2.84	3.28	3.88	4.39	4.93
	41	1.77	2.14	2.80	3.23	3.82	4.32	4.86
	42	1.74	2.10	2.75	3.19	3.77	4.26	4.79
	43	1.71	2.07	2.71	3.14	3.71	4.21	4.73
	44	1.68	2.04	2.67	3.10	3.66	4.15	4.67
0.750	45	1.66	2.01	2.64	3.05	3.61	4.09	4.61
	46	1.63	1.98	2.60	3.01	3.56	4.04	4.55
	47	1.61	1.95	2.56	2.97	3.52	3.99	4.49
	48	1.58	1.92	2.53	2.93	3.47	3.94	4.43
	49	1.56	1.89	2.50	2.89	3.43	3.89	4.38
	50	1.54	1.87	2.46	2.86	3.38	3.84	4.33
	51	1.52	1.84	2.43	2.82	3.34	3.79	4.27
	52	1.50	1.82	2.40	2.78	3.30	3.75	4.22
	53	1.48	1.80	2.37	2.75	3.26	3.70	4.18
	54	1.46	1.77	2.34	2.72	3.22	3.66	4.13
	55	1.44	1.75	2.31	2.69	3.19	3.62	4.08
	56	1.42	1.73	2.28	2.65	3.15	3.58	4.04
	57	1.40	1.71	2.26	2.62	3.11	3.54	3.99
	58	1.39	1.69	2.23	2.59	3.08	3.50	3.95
	59	1.37	1.67	2.21	2.56	3.04	3.46	3.91
1	60	1.35	1.65	2.18	2.54	3.01	3.42	3.87
2	120	0.81	0.99	1.33	1.56	1.88	2.15	2.44
3	180	0.59	0.73	0.98	1.16	1.40	1.60	1.83
6	360	0.34	0.42	0.58	0.68	0.83	0.96	1.10
12	720	0.19	0.24	0.33	0.40	0.49	0.56	0.65
24	1440	0.11	0.14	0.19	0.23	0.28	0.33	0.38

Table A-10 Kaufman County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.82108	0.80451	0.79370	0.78786	0.77191	0.76117	0.75823
	b	47.296	54.125	68.520	78.197	86.089	92.329	103.645
	d	9	10	12	13	13	13	14
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.42	6.13	7.23	8.02	9.25	10.23	11.12
	6	5.12	5.82	6.91	7.69	8.87	9.82	10.69
	7	4.85	5.54	6.62	7.38	8.52	9.44	10.30
	8	4.62	5.29	6.36	7.10	8.21	9.10	9.95
	9	4.41	5.07	6.11	6.85	7.92	8.78	9.62
	10	4.22	4.86	5.89	6.61	7.65	8.49	9.31
	11	4.04	4.67	5.69	6.39	7.41	8.22	9.03
	12	3.88	4.50	5.50	6.19	7.18	7.97	8.76
	13	3.74	4.34	5.32	6.00	6.96	7.73	8.52
	14	3.60	4.20	5.16	5.83	6.76	7.51	8.28
0.250	15	3.48	4.06	5.01	5.66	6.57	7.31	8.07
	16	3.37	3.94	4.87	5.51	6.40	7.12	7.86
	17	3.26	3.82	4.73	5.36	6.23	6.93	7.67
	18	3.16	3.71	4.61	5.23	6.08	6.76	7.49
	19	3.07	3.60	4.49	5.10	5.93	6.60	7.31
	20	2.98	3.51	4.38	4.98	5.79	6.45	7.15
	21	2.90	3.42	4.27	4.86	5.66	6.30	6.99
	22	2.82	3.33	4.17	4.75	5.53	6.17	6.85
	23	2.75	3.25	4.08	4.65	5.42	6.04	6.71
	24	2.68	3.17	3.99	4.55	5.30	5.91	6.57
	25	2.61	3.10	3.90	4.45	5.19	5.79	6.44
	26	2.55	3.03	3.82	4.36	5.09	5.68	6.32
	27	2.49	2.96	3.74	4.28	4.99	5.57	6.20
	28	2.44	2.90	3.67	4.19	4.90	5.47	6.09
	29	2.39	2.84	3.60	4.11	4.81	5.37	5.98
0.500	30	2.34	2.78	3.53	4.04	4.72	5.27	5.88
	31	2.29	2.73	3.46	3.97	4.64	5.18	5.78
	32	2.24	2.68	3.40	3.90	4.56	5.09	5.69
	33	2.20	2.63	3.34	3.83	4.48	5.01	5.59
	34	2.16	2.58	3.28	3.77	4.41	4.93	5.51
	35	2.12	2.53	3.23	3.70	4.34	4.85	5.42
	36	2.08	2.49	3.17	3.64	4.27	4.77	5.34
	37	2.04	2.44	3.12	3.59	4.20	4.70	5.26
	38	2.00	2.40	3.07	3.53	4.14	4.63	5.18
	39	1.97	2.36	3.02	3.48	4.08	4.56	5.11
	40	1.94	2.33	2.98	3.43	4.02	4.50	5.03
	41	1.90	2.29	2.93	3.38	3.96	4.43	4.97
	42	1.87	2.25	2.89	3.33	3.90	4.37	4.90
	43	1.84	2.22	2.85	3.28	3.85	4.31	4.83
	44	1.82	2.19	2.81	3.23	3.80	4.25	4.77
0.750	45	1.79	2.15	2.77	3.19	3.75	4.20	4.71
	46	1.76	2.12	2.73	3.15	3.70	4.14	4.65
	47	1.74	2.09	2.69	3.11	3.65	4.09	4.59
	48	1.71	2.06	2.66	3.07	3.60	4.04	4.53
	49	1.69	2.04	2.62	3.03	3.56	3.99	4.48
	50	1.66	2.01	2.59	2.99	3.52	3.94	4.43
	51	1.64	1.98	2.56	2.95	3.47	3.90	4.37
	52	1.62	1.96	2.52	2.92	3.43	3.85	4.32
	53	1.60	1.93	2.49	2.88	3.39	3.81	4.28
	54	1.58	1.91	2.46	2.85	3.35	3.76	4.23
	55	1.56	1.88	2.43	2.81	3.31	3.72	4.18
	56	1.54	1.86	2.41	2.78	3.28	3.68	4.14
	57	1.52	1.84	2.38	2.75	3.24	3.64	4.09
	58	1.50	1.82	2.35	2.72	3.21	3.60	4.05
	59	1.48	1.79	2.33	2.69	3.17	3.56	4.01
1	60	1.46	1.77	2.30	2.66	3.14	3.52	3.96
2	120	0.87	1.08	1.42	1.66	1.97	2.23	2.53
3	180	0.64	0.79	1.06	1.24	1.48	1.68	1.91
6	360	0.37	0.46	0.62	0.74	0.89	1.02	1.16
12	720	0.21	0.27	0.36	0.43	0.53	0.61	0.70
24	1440	0.12	0.15	0.21	0.25	0.31	0.36	0.41

Table A-11 Navarro County

	Coefficients	Return Period (Years) 1	2	5	10	25	50	100
	e	0.82920	0.80921	0.80665	0.79885	0.79034	0.78696	0.78204
	b	49.070	55.346	74.472	84.510	98.346	111.208	124.111
	d	9	10	13	14	15	16	17
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.50	6.19	7.23	8.04	9.22	10.13	11.07
	6	5.20	5.87	6.93	7.72	8.87	9.77	10.69
	7	4.92	5.59	6.65	7.42	8.55	9.43	10.34
	8	4.68	5.34	6.39	7.15	8.25	9.12	10.01
	9	4.47	5.11	6.15	6.90	7.98	8.83	9.71
	10	4.27	4.90	5.94	6.67	7.73	8.56	9.43
	11	4.09	4.71	5.74	6.46	7.49	8.31	9.16
	12	3.93	4.54	5.55	6.26	7.27	8.08	8.92
	13	3.78	4.38	5.38	6.07	7.06	7.86	8.68
	14	3.64	4.23	5.22	5.90	6.87	7.65	8.46
0.250	15	3.52	4.09	5.07	5.74	6.69	7.46	8.26
	16	3.40	3.96	4.92	5.58	6.52	7.27	8.06
	17	3.29	3.84	4.79	5.44	6.36	7.10	7.87
	18	3.19	3.73	4.67	5.30	6.20	6.93	7.70
	19	3.10	3.63	4.55	5.17	6.06	6.78	7.53
	20	3.01	3.53	4.44	5.05	5.92	6.63	7.37
	21	2.92	3.44	4.33	4.94	5.79	6.49	7.22
	22	2.85	3.35	4.23	4.83	5.67	6.35	7.07
	23	2.77	3.27	4.14	4.72	5.55	6.22	6.93
	24	2.70	3.19	4.05	4.62	5.44	6.10	6.80
	25	2.64	3.12	3.96	4.53	5.33	5.98	6.67
	26	2.57	3.05	3.88	4.44	5.23	5.87	6.55
	27	2.51	2.98	3.80	4.35	5.13	5.76	6.44
	28	2.46	2.92	3.72	4.27	5.03	5.66	6.32
	29	2.40	2.85	3.65	4.19	4.94	5.56	6.22
0.500	30	2.35	2.80	3.58	4.11	4.85	5.47	6.11
	31	2.30	2.74	3.52	4.04	4.77	5.37	6.01
	32	2.26	2.69	3.45	3.97	4.69	5.29	5.92
	33	2.21	2.64	3.39	3.90	4.61	5.20	5.82
	34	2.17	2.59	3.34	3.84	4.54	5.12	5.73
	35	2.13	2.54	3.28	3.77	4.47	5.04	5.65
	36	2.09	2.50	3.23	3.71	4.40	4.96	5.56
	37	2.05	2.45	3.17	3.65	4.33	4.89	5.48
	38	2.02	2.41	3.12	3.60	4.27	4.82	5.40
	39	1.98	2.37	3.07	3.54	4.20	4.75	5.33
	40	1.95	2.33	3.03	3.49	4.14	4.68	5.26
	41	1.91	2.30	2.98	3.44	4.08	4.62	5.18
	42	1.88	2.26	2.94	3.39	4.03	4.55	5.12
	43	1.85	2.23	2.90	3.34	3.97	4.49	5.05
	44	1.82	2.19	2.86	3.30	3.92	4.43	4.98
0.750	45	1.80	2.16	2.82	3.25	3.87	4.38	4.92
	46	1.77	2.13	2.78	3.21	3.82	4.32	4.86
	47	1.74	2.10	2.74	3.17	3.77	4.27	4.80
	48	1.72	2.07	2.70	3.13	3.72	4.21	4.74
	49	1.69	2.04	2.67	3.09	3.68	4.16	4.69
	50	1.67	2.01	2.63	3.05	3.63	4.11	4.63
	51	1.65	1.99	2.60	3.01	3.59	4.07	4.58
	52	1.62	1.96	2.57	2.97	3.54	4.02	4.53
	53	1.60	1.94	2.54	2.94	3.50	3.97	4.48
	54	1.58	1.91	2.51	2.90	3.46	3.93	4.43
	55	1.56	1.89	2.48	2.87	3.42	3.88	4.38
	56	1.54	1.87	2.45	2.84	3.39	3.84	4.33
	57	1.52	1.84	2.42	2.81	3.35	3.80	4.29
	58	1.50	1.82	2.39	2.77	3.31	3.76	4.24
	59	1.48	1.80	2.36	2.74	3.28	3.72	4.20
1	60	1.47	1.78	2.34	2.71	3.24	3.68	4.15
2	120	0.87	1.08	1.44	1.69	2.04	2.33	2.65
3	180	0.64	0.79	1.07	1.26	1.52	1.75	1.99
6	360	0.36	0.46	0.63	0.74	0.91	1.05	1.20
12	720	0.21	0.27	0.36	0.43	0.53	0.62	0.71
24	1440	0.12	0.15	0.21	0.25	0.31	0.36	0.42

Table A-12 Palo Pinto County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.82438	0.81924	0.82409	0.81899	0.81251	0.80950	0.80567
	b	42.135	50.817	71.566	82.741	97.985	111.741	126.255
	d	8	9	12	13	14	15	16
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.09	5.85	6.93	7.76	8.96	9.89	10.86
	6	4.78	5.53	6.61	7.42	8.59	9.50	10.46
	7	4.52	5.24	6.32	7.12	8.26	9.15	10.10
	8	4.29	4.99	6.06	6.84	7.95	8.83	9.76
	9	4.08	4.76	5.82	6.58	7.67	8.53	9.44
	10	3.89	4.55	5.60	6.35	7.41	8.25	9.15
	11	3.72	4.37	5.40	6.13	7.17	7.99	8.87
	12	3.57	4.20	5.22	5.93	6.94	7.75	8.62
	13	3.42	4.04	5.04	5.74	6.73	7.53	8.38
	14	3.30	3.89	4.88	5.56	6.54	7.32	8.15
0.250	15	3.18	3.76	4.73	5.40	6.35	7.12	7.94
	16	3.07	3.64	4.59	5.25	6.18	6.93	7.74
	17	2.97	3.52	4.46	5.10	6.02	6.76	7.55
	18	2.87	3.41	4.34	4.97	5.86	6.59	7.37
	19	2.78	3.31	4.22	4.84	5.72	6.43	7.20
	20	2.70	3.22	4.11	4.72	5.58	6.28	7.04
	21	2.62	3.13	4.01	4.61	5.45	6.14	6.88
	22	2.55	3.05	3.91	4.50	5.33	6.01	6.74
	23	2.48	2.97	3.82	4.40	5.21	5.88	6.60
	24	2.42	2.90	3.73	4.30	5.10	5.76	6.46
	25	2.36	2.83	3.65	4.21	4.99	5.64	6.34
	26	2.30	2.76	3.57	4.12	4.89	5.53	6.22
	27	2.25	2.70	3.50	4.03	4.79	5.42	6.10
	28	2.20	2.64	3.42	3.95	4.70	5.32	5.99
	29	2.15	2.58	3.35	3.88	4.61	5.22	5.88
0.500	30	2.10	2.53	3.29	3.80	4.53	5.13	5.78
	31	2.06	2.47	3.23	3.73	4.45	5.04	5.68
	32	2.01	2.43	3.16	3.66	4.37	4.95	5.58
	33	1.97	2.38	3.11	3.60	4.29	4.87	5.49
	34	1.93	2.33	3.05	3.53	4.22	4.79	5.40
	35	1.90	2.29	3.00	3.47	4.15	4.71	5.32
	36	1.86	2.25	2.95	3.42	4.08	4.63	5.23
	37	1.83	2.21	2.90	3.36	4.02	4.56	5.15
	38	1.79	2.17	2.85	3.31	3.95	4.49	5.08
	39	1.76	2.13	2.80	3.25	3.89	4.42	5.00
	40	1.73	2.10	2.76	3.20	3.83	4.36	4.93
	41	1.70	2.06	2.71	3.15	3.78	4.30	4.86
	42	1.68	2.03	2.67	3.11	3.72	4.23	4.79
	43	1.65	2.00	2.63	3.06	3.67	4.18	4.73
	44	1.62	1.97	2.59	3.02	3.62	4.12	4.66
0.750	45	1.60	1.94	2.56	2.98	3.57	4.06	4.60
	46	1.57	1.91	2.52	2.93	3.52	4.01	4.54
	47	1.55	1.88	2.49	2.89	3.47	3.96	4.48
	48	1.53	1.85	2.45	2.85	3.43	3.91	4.43
	49	1.50	1.83	2.42	2.82	3.38	3.86	4.37
	50	1.48	1.80	2.39	2.78	3.34	3.81	4.32
	51	1.46	1.78	2.35	2.74	3.30	3.76	4.27
	52	1.44	1.75	2.32	2.71	3.26	3.72	4.22
	53	1.42	1.73	2.29	2.68	3.22	3.67	4.17
	54	1.40	1.71	2.27	2.64	3.18	3.63	4.12
	55	1.38	1.68	2.24	2.61	3.14	3.59	4.07
	56	1.37	1.66	2.21	2.58	3.10	3.54	4.03
	57	1.35	1.64	2.18	2.55	3.07	3.51	3.98
	58	1.33	1.62	2.16	2.52	3.03	3.47	3.94
	59	1.32	1.60	2.13	2.49	3.00	3.43	3.90
1	60	1.30	1.58	2.11	2.46	2.97	3.39	3.85
2	120	0.77	0.95	1.28	1.51	1.83	2.11	2.41
3	180	0.56	0.69	0.94	1.11	1.36	1.56	1.80
6	360	0.32	0.40	0.54	0.65	0.80	0.92	1.06
12	720	0.18	0.23	0.31	0.37	0.46	0.53	0.62
24	1440	0.10	0.13	0.18	0.21	0.26	0.31	0.36

Table A-13 Parker County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.81993	0.81528	0.80996	0.80658	0.80148	0.80055	0.79789
	b	42.333	51.064	67.052	77.954	92.557	106.196	120.205
	d	8	9	11	12	13	14	15
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.17	5.94	7.10	7.93	9.13	10.06	11.01
	6	4.86	5.61	6.76	7.57	8.74	9.65	10.59
	7	4.60	5.33	6.45	7.25	8.39	9.28	10.21
	8	4.36	5.07	6.18	6.96	8.07	8.94	9.85
	9	4.15	4.84	5.92	6.69	7.77	8.63	9.52
	10	3.96	4.63	5.69	6.44	7.50	8.34	9.22
	11	3.79	4.44	5.48	6.22	7.25	8.07	8.93
	12	3.63	4.27	5.29	6.01	7.01	7.82	8.67
	13	3.49	4.11	5.11	5.81	6.80	7.59	8.42
	14	3.36	3.96	4.94	5.63	6.59	7.37	8.19
0.250	15	3.24	3.83	4.79	5.46	6.41	7.17	7.97
	16	3.13	3.70	4.65	5.30	6.23	6.98	7.76
	17	3.02	3.59	4.51	5.16	6.06	6.80	7.57
	18	2.93	3.48	4.38	5.02	5.90	6.62	7.38
	19	2.84	3.38	4.27	4.89	5.76	6.46	7.21
	20	2.75	3.28	4.15	4.76	5.61	6.31	7.05
	21	2.68	3.19	4.05	4.65	5.48	6.17	6.89
	22	2.60	3.11	3.95	4.54	5.36	6.03	6.74
	23	2.53	3.03	3.85	4.43	5.24	5.90	6.60
	24	2.47	2.95	3.77	4.33	5.12	5.77	6.46
	25	2.41	2.88	3.68	4.24	5.01	5.65	6.33
	26	2.35	2.81	3.60	4.15	4.91	5.54	6.21
	27	2.29	2.75	3.52	4.06	4.81	5.43	6.09
	28	2.24	2.69	3.45	3.98	4.72	5.33	5.98
	29	2.19	2.63	3.38	3.90	4.63	5.23	5.87
0.500	30	2.14	2.58	3.31	3.82	4.54	5.13	5.77
	31	2.10	2.52	3.25	3.75	4.46	5.04	5.67
	32	2.06	2.47	3.19	3.68	4.38	4.95	5.57
	33	2.02	2.43	3.13	3.62	4.30	4.87	5.48
	34	1.98	2.38	3.07	3.55	4.23	4.79	5.39
	35	1.94	2.33	3.02	3.49	4.16	4.71	5.30
	36	1.90	2.29	2.97	3.43	4.09	4.63	5.22
	37	1.87	2.25	2.92	3.38	4.02	4.56	5.14
	38	1.83	2.21	2.87	3.32	3.96	4.49	5.06
	39	1.80	2.17	2.82	3.27	3.90	4.42	4.98
	40	1.77	2.14	2.78	3.22	3.84	4.36	4.91
	41	1.74	2.10	2.73	3.17	3.78	4.29	4.84
	42	1.71	2.07	2.69	3.12	3.73	4.23	4.77
	43	1.68	2.04	2.65	3.08	3.68	4.17	4.71
	44	1.66	2.01	2.61	3.03	3.62	4.12	4.64
0.750	45	1.63	1.98	2.57	2.99	3.57	4.06	4.58
	46	1.61	1.95	2.54	2.95	3.52	4.01	4.52
	47	1.58	1.92	2.50	2.91	3.48	3.95	4.46
	48	1.56	1.89	2.47	2.87	3.43	3.90	4.41
	49	1.54	1.86	2.43	2.83	3.39	3.85	4.35
	50	1.52	1.84	2.40	2.79	3.34	3.80	4.30
	51	1.50	1.81	2.37	2.76	3.30	3.76	4.25
	52	1.47	1.79	2.34	2.72	3.26	3.71	4.20
	53	1.45	1.77	2.31	2.69	3.22	3.67	4.15
	54	1.44	1.74	2.28	2.66	3.18	3.62	4.10
	55	1.42	1.72	2.25	2.62	3.15	3.58	4.05
	56	1.40	1.70	2.23	2.59	3.11	3.54	4.01
	57	1.38	1.68	2.20	2.56	3.07	3.50	3.96
	58	1.36	1.66	2.17	2.53	3.04	3.46	3.92
	59	1.35	1.64	2.15	2.50	3.00	3.42	3.88
1	60	1.33	1.62	2.12	2.48	2.97	3.39	3.84
2	120	0.79	0.97	1.29	1.52	1.84	2.10	2.40
3	180	0.58	0.71	0.95	1.12	1.36	1.57	1.79
6	360	0.33	0.41	0.56	0.66	0.80	0.93	1.06
12	720	0.19	0.24	0.32	0.38	0.47	0.54	0.62
24	1440	0.11	0.14	0.18	0.22	0.27	0.31	0.36

Table A-14 Rockwall County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.83417	0.80556	0.79987	0.79563	0.78076	0.76899	0.75652
	b	50.825	53.943	69.378	79.520	87.731	93.486	98.870
	d	10	10	12	13	13	13	13
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.31	6.09	7.19	7.98	9.19	10.13	11.10
	6	5.03	5.78	6.87	7.64	8.81	9.71	10.66
	7	4.78	5.50	6.58	7.33	8.46	9.34	10.25
	8	4.56	5.26	6.32	7.05	8.14	8.99	9.88
	9	4.36	5.03	6.08	6.80	7.85	8.68	9.54
	10	4.18	4.83	5.85	6.56	7.59	8.39	9.22
	11	4.01	4.64	5.65	6.34	7.34	8.12	8.93
	12	3.86	4.47	5.46	6.14	7.11	7.87	8.66
	13	3.72	4.31	5.29	5.95	6.89	7.63	8.41
	14	3.59	4.17	5.12	5.78	6.69	7.41	8.17
0.250	15	3.47	4.03	4.97	5.61	6.51	7.21	7.95
	16	3.36	3.91	4.83	5.46	6.33	7.02	7.74
	17	3.25	3.79	4.69	5.31	6.16	6.84	7.54
	18	3.15	3.68	4.57	5.17	6.01	6.67	7.36
	19	3.06	3.58	4.45	5.05	5.86	6.51	7.18
	20	2.98	3.48	4.34	4.92	5.72	6.35	7.02
	21	2.90	3.39	4.23	4.81	5.59	6.21	6.86
	22	2.82	3.31	4.13	4.70	5.47	6.07	6.71
	23	2.75	3.23	4.04	4.59	5.35	5.94	6.57
	24	2.68	3.15	3.95	4.50	5.23	5.82	6.44
	25	2.62	3.08	3.86	4.40	5.13	5.70	6.31
	26	2.56	3.01	3.78	4.31	5.02	5.59	6.19
	27	2.50	2.94	3.70	4.23	4.92	5.48	6.07
	28	2.44	2.88	3.63	4.14	4.83	5.38	5.96
	29	2.39	2.82	3.56	4.06	4.74	5.28	5.85
0.500	30	2.34	2.76	3.49	3.99	4.65	5.18	5.75
	31	2.29	2.71	3.43	3.92	4.57	5.09	5.65
	32	2.25	2.66	3.36	3.85	4.49	5.01	5.55
	33	2.21	2.61	3.30	3.78	4.42	4.92	5.46
	34	2.16	2.56	3.25	3.72	4.34	4.84	5.37
	35	2.12	2.51	3.19	3.65	4.27	4.76	5.29
	36	2.08	2.47	3.14	3.60	4.20	4.69	5.20
	37	2.05	2.43	3.09	3.54	4.14	4.62	5.13
	38	2.01	2.39	3.04	3.48	4.07	4.55	5.05
	39	1.98	2.35	2.99	3.43	4.01	4.48	4.98
	40	1.94	2.31	2.94	3.38	3.95	4.41	4.90
	41	1.91	2.27	2.90	3.33	3.90	4.35	4.84
	42	1.88	2.24	2.85	3.28	3.84	4.29	4.77
	43	1.85	2.20	2.81	3.23	3.79	4.23	4.70
	44	1.82	2.17	2.77	3.19	3.73	4.17	4.64
0.750	45	1.80	2.14	2.73	3.14	3.68	4.12	4.58
	46	1.77	2.11	2.70	3.10	3.64	4.06	4.52
	47	1.74	2.08	2.66	3.06	3.59	4.01	4.47
	48	1.72	2.05	2.62	3.02	3.54	3.96	4.41
	49	1.69	2.02	2.59	2.98	3.50	3.91	4.36
	50	1.67	1.99	2.56	2.94	3.45	3.86	4.30
	51	1.65	1.97	2.52	2.91	3.41	3.82	4.25
	52	1.63	1.94	2.49	2.87	3.37	3.77	4.20
	53	1.60	1.92	2.46	2.84	3.33	3.73	4.15
	54	1.58	1.89	2.43	2.80	3.29	3.69	4.11
	55	1.56	1.87	2.40	2.77	3.25	3.64	4.06
	56	1.54	1.85	2.37	2.74	3.22	3.60	4.02
	57	1.52	1.82	2.35	2.71	3.18	3.56	3.97
	58	1.50	1.80	2.32	2.68	3.15	3.52	3.93
	59	1.49	1.78	2.29	2.65	3.11	3.49	3.89
1	60	1.47	1.76	2.27	2.62	3.08	3.45	3.85
2	120	0.88	1.07	1.40	1.62	1.93	2.18	2.45
3	180	0.64	0.79	1.03	1.21	1.44	1.63	1.84
6	360	0.37	0.46	0.61	0.72	0.86	0.98	1.12
12	720	0.21	0.27	0.35	0.42	0.51	0.59	0.67
24	1440	0.12	0.15	0.21	0.24	0.30	0.35	0.40

Table A-15 Somervell County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.81907	0.81396	0.80585	0.80055	0.79375	0.78198	0.77885
	b	42.170	50.658	65.716	75.784	89.151	95.704	108.069
	d	8	9	11	12	13	13	14
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.16	5.91	7.04	7.84	8.99	9.98	10.91
	6	4.86	5.59	6.70	7.49	8.61	9.57	10.48
	7	4.59	5.30	6.40	7.18	8.27	9.19	10.09
	8	4.35	5.05	6.13	6.89	7.95	8.85	9.73
	9	4.14	4.82	5.88	6.62	7.67	8.53	9.40
	10	3.95	4.61	5.65	6.38	7.40	8.24	9.09
	11	3.78	4.42	5.44	6.16	7.15	7.97	8.81
	12	3.63	4.25	5.25	5.95	6.93	7.72	8.54
	13	3.48	4.09	5.07	5.76	6.71	7.49	8.30
	14	3.35	3.95	4.91	5.58	6.52	7.27	8.06
0.250	15	3.23	3.81	4.76	5.42	6.33	7.07	7.85
	16	3.12	3.69	4.62	5.26	6.16	6.88	7.64
	17	3.02	3.57	4.48	5.12	5.99	6.70	7.45
	18	2.92	3.46	4.36	4.98	5.84	6.53	7.27
	19	2.84	3.36	4.24	4.85	5.69	6.37	7.10
	20	2.75	3.27	4.13	4.73	5.56	6.22	6.93
	21	2.67	3.18	4.02	4.61	5.43	6.07	6.78
	22	2.60	3.10	3.93	4.50	5.30	5.94	6.63
	23	2.53	3.02	3.83	4.40	5.19	5.81	6.49
	24	2.47	2.94	3.74	4.30	5.07	5.68	6.36
	25	2.41	2.87	3.66	4.21	4.97	5.57	6.23
	26	2.35	2.80	3.58	4.12	4.87	5.45	6.11
	27	2.29	2.74	3.50	4.04	4.77	5.35	5.99
	28	2.24	2.68	3.43	3.95	4.68	5.25	5.88
	29	2.19	2.62	3.36	3.88	4.59	5.15	5.77
0.500	30	2.14	2.57	3.30	3.80	4.50	5.05	5.67
	31	2.10	2.52	3.23	3.73	4.42	4.96	5.57
	32	2.05	2.47	3.17	3.66	4.34	4.88	5.48
	33	2.01	2.42	3.11	3.60	4.27	4.79	5.39
	34	1.97	2.37	3.06	3.54	4.20	4.71	5.30
	35	1.94	2.33	3.00	3.48	4.13	4.64	5.22
	36	1.90	2.29	2.95	3.42	4.06	4.56	5.13
	37	1.87	2.25	2.90	3.36	4.00	4.49	5.06
	38	1.83	2.21	2.86	3.31	3.93	4.42	4.98
	39	1.80	2.17	2.81	3.26	3.87	4.36	4.91
	40	1.77	2.13	2.76	3.20	3.81	4.29	4.84
	41	1.74	2.10	2.72	3.16	3.76	4.23	4.77
	42	1.71	2.06	2.68	3.11	3.70	4.17	4.70
	43	1.68	2.03	2.64	3.06	3.65	4.11	4.64
	44	1.66	2.00	2.60	3.02	3.60	4.05	4.57
0.750	45	1.63	1.97	2.56	2.98	3.55	4.00	4.51
	46	1.61	1.94	2.53	2.94	3.50	3.95	4.45
	47	1.58	1.91	2.49	2.90	3.46	3.89	4.40
	48	1.56	1.89	2.46	2.86	3.41	3.84	4.34
	49	1.54	1.86	2.43	2.82	3.37	3.80	4.29
	50	1.52	1.83	2.39	2.78	3.33	3.75	4.24
	51	1.49	1.81	2.36	2.75	3.28	3.70	4.19
	52	1.47	1.78	2.33	2.71	3.24	3.66	4.14
	53	1.45	1.76	2.30	2.68	3.21	3.61	4.09
	54	1.44	1.74	2.27	2.65	3.17	3.57	4.04
	55	1.42	1.72	2.25	2.62	3.13	3.53	4.00
	56	1.40	1.69	2.22	2.59	3.09	3.49	3.95
	57	1.38	1.67	2.19	2.56	3.06	3.45	3.91
	58	1.36	1.65	2.17	2.53	3.02	3.41	3.86
	59	1.35	1.63	2.14	2.50	2.99	3.38	3.82
1	60	1.33	1.61	2.12	2.47	2.96	3.34	3.78
2	120	0.79	0.97	1.29	1.52	1.84	2.09	2.38
3	180	0.58	0.71	0.95	1.13	1.37	1.56	1.79
6	360	0.33	0.41	0.56	0.66	0.81	0.93	1.07
12	720	0.19	0.24	0.32	0.39	0.47	0.55	0.63
24	1440	0.11	0.14	0.19	0.22	0.28	0.32	0.37

Table A-16 Tarrant County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.82169	0.81144	0.81423	0.79952	0.79381	0.78265	0.77982
	b	43.653	51.393	71.154	77.103	90.982	97.721	110.202
	d	8	9	12	12	13	13	14
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.31	6.04	7.08	8.00	9.17	10.18	11.09
	6	4.99	5.71	6.76	7.65	8.79	9.75	10.66
	7	4.72	5.42	6.47	7.32	8.44	9.37	10.26
	8	4.47	5.16	6.21	7.03	8.12	9.02	9.89
	9	4.26	4.92	5.97	6.76	7.82	8.70	9.56
	10	4.06	4.71	5.74	6.51	7.55	8.40	9.24
	11	3.88	4.52	5.54	6.29	7.30	8.12	8.95
	12	3.72	4.35	5.35	6.08	7.07	7.87	8.68
	13	3.58	4.18	5.18	5.88	6.85	7.63	8.43
	14	3.44	4.04	5.01	5.70	6.65	7.41	8.20
0.250	15	3.32	3.90	4.86	5.53	6.46	7.20	7.98
	16	3.21	3.77	4.72	5.37	6.28	7.01	7.77
	17	3.10	3.65	4.59	5.22	6.12	6.82	7.57
	18	3.00	3.54	4.46	5.08	5.96	6.65	7.39
	19	2.91	3.44	4.34	4.95	5.81	6.49	7.21
	20	2.82	3.34	4.23	4.83	5.67	6.33	7.05
	21	2.74	3.25	4.13	4.71	5.54	6.19	6.89
	22	2.67	3.17	4.03	4.60	5.41	6.05	6.74
	23	2.60	3.09	3.94	4.49	5.29	5.91	6.60
	24	2.53	3.01	3.85	4.39	5.18	5.79	6.46
	25	2.47	2.94	3.76	4.30	5.07	5.67	6.33
	26	2.41	2.87	3.68	4.21	4.97	5.56	6.21
	27	2.35	2.81	3.60	4.12	4.87	5.45	6.09
	28	2.30	2.74	3.53	4.04	4.77	5.34	5.98
	29	2.25	2.69	3.46	3.96	4.68	5.24	5.87
0.500	30	2.20	2.63	3.39	3.88	4.60	5.15	5.76
	31	2.15	2.58	3.33	3.81	4.51	5.06	5.66
	32	2.11	2.52	3.27	3.74	4.43	4.97	5.57
	33	2.06	2.48	3.21	3.68	4.36	4.88	5.47
	34	2.02	2.43	3.15	3.61	4.28	4.80	5.38
	35	1.99	2.38	3.10	3.55	4.21	4.72	5.30
	36	1.95	2.34	3.04	3.49	4.14	4.65	5.22
	37	1.91	2.30	2.99	3.43	4.08	4.57	5.14
	38	1.88	2.26	2.94	3.38	4.01	4.50	5.06
	39	1.85	2.22	2.90	3.33	3.95	4.44	4.98
	40	1.81	2.18	2.85	3.27	3.89	4.37	4.91
	41	1.78	2.15	2.81	3.22	3.84	4.31	4.84
	42	1.75	2.11	2.76	3.18	3.78	4.25	4.77
	43	1.73	2.08	2.72	3.13	3.73	4.19	4.71
	44	1.70	2.05	2.68	3.09	3.67	4.13	4.65
0.750	45	1.67	2.02	2.65	3.04	3.62	4.07	4.58
	46	1.65	1.99	2.61	3.00	3.57	4.02	4.52
	47	1.62	1.96	2.57	2.96	3.53	3.97	4.47
	48	1.60	1.93	2.54	2.92	3.48	3.91	4.41
	49	1.57	1.91	2.50	2.88	3.44	3.87	4.36
	50	1.55	1.88	2.47	2.84	3.39	3.82	4.30
	51	1.53	1.85	2.44	2.81	3.35	3.77	4.25
	52	1.51	1.83	2.41	2.77	3.31	3.72	4.20
	53	1.49	1.81	2.38	2.74	3.27	3.68	4.15
	54	1.47	1.78	2.35	2.71	3.23	3.64	4.10
	55	1.45	1.76	2.32	2.67	3.19	3.60	4.06
	56	1.43	1.74	2.29	2.64	3.16	3.55	4.01
	57	1.41	1.72	2.26	2.61	3.12	3.51	3.97
	58	1.40	1.69	2.24	2.58	3.09	3.48	3.92
	59	1.38	1.67	2.21	2.55	3.05	3.44	3.88
1	60	1.36	1.65	2.19	2.52	3.02	3.40	3.84
2	120	0.81	1.00	1.34	1.55	1.88	2.13	2.42
3	180	0.59	0.73	0.98	1.15	1.40	1.59	1.81
6	360	0.34	0.42	0.57	0.68	0.83	0.95	1.09
12	720	0.19	0.24	0.33	0.40	0.48	0.56	0.64
24	1440	0.11	0.14	0.19	0.23	0.28	0.33	0.38

Table A-17 Wise County

		Return Period (Years)						
	Coefficients	1	2	5	10	25	50	100
	e	0.80578	0.79881	0.79496	0.79143	0.78680	0.78570	0.78307
	b	38.593	46.352	61.396	71.487	85.260	97.989	111.129
	d	7	8	10	11	12	13	14
Hours	Minutes	Rainfall Intensity (inches per hour)						
0.083	5	5.21	5.97	7.13	7.97	9.18	10.11	11.08
	6	4.89	5.63	6.78	7.59	8.77	9.69	10.64
	7	4.60	5.33	6.46	7.26	8.41	9.31	10.24
	8	4.35	5.06	6.17	6.95	8.07	8.96	9.88
	9	4.13	4.82	5.91	6.68	7.77	8.64	9.54
	10	3.94	4.61	5.67	6.42	7.49	8.34	9.23
	11	3.76	4.41	5.46	6.19	7.23	8.07	8.94
	12	3.60	4.23	5.26	5.98	7.00	7.81	8.67
	13	3.45	4.07	5.08	5.78	6.77	7.58	8.41
	14	3.32	3.92	4.91	5.60	6.57	7.35	8.18
0.250	15	3.20	3.79	4.75	5.42	6.38	7.15	7.96
	16	3.09	3.66	4.61	5.27	6.20	6.95	7.75
	17	2.98	3.54	4.47	5.12	6.03	6.77	7.55
	18	2.88	3.43	4.34	4.98	5.87	6.60	7.37
	19	2.79	3.33	4.22	4.84	5.72	6.44	7.19
	20	2.71	3.24	4.11	4.72	5.58	6.28	7.02
	21	2.63	3.15	4.00	4.60	5.44	6.14	6.87
	22	2.56	3.06	3.90	4.49	5.32	6.00	6.72
	23	2.49	2.98	3.81	4.39	5.20	5.87	6.57
	24	2.43	2.91	3.72	4.29	5.08	5.74	6.44
	25	2.36	2.84	3.64	4.19	4.98	5.62	6.31
	26	2.31	2.77	3.56	4.10	4.87	5.51	6.18
	27	2.25	2.71	3.48	4.02	4.77	5.40	6.07
	28	2.20	2.65	3.41	3.94	4.68	5.30	5.95
	29	2.15	2.59	3.34	3.86	4.59	5.20	5.84
0.500	30	2.10	2.54	3.27	3.78	4.50	5.10	5.74
	31	2.06	2.48	3.21	3.71	4.42	5.01	5.64
	32	2.02	2.43	3.15	3.64	4.34	4.92	5.54
	33	1.98	2.39	3.09	3.58	4.27	4.84	5.45
	34	1.94	2.34	3.03	3.51	4.19	4.76	5.36
	35	1.90	2.30	2.98	3.45	4.12	4.68	5.28
	36	1.86	2.26	2.93	3.40	4.05	4.60	5.19
	37	1.83	2.22	2.88	3.34	3.99	4.53	5.11
	38	1.80	2.18	2.83	3.29	3.93	4.46	5.04
	39	1.76	2.14	2.78	3.23	3.87	4.39	4.96
	40	1.73	2.10	2.74	3.18	3.81	4.33	4.89
	41	1.71	2.07	2.70	3.13	3.75	4.27	4.82
	42	1.68	2.04	2.65	3.09	3.70	4.21	4.75
	43	1.65	2.00	2.61	3.04	3.64	4.15	4.69
	44	1.62	1.97	2.58	3.00	3.59	4.09	4.62
0.750	45	1.60	1.94	2.54	2.96	3.54	4.03	4.56
	46	1.57	1.92	2.50	2.91	3.49	3.98	4.50
	47	1.55	1.89	2.47	2.87	3.45	3.93	4.44
	48	1.53	1.86	2.43	2.84	3.40	3.88	4.39
	49	1.51	1.83	2.40	2.80	3.36	3.83	4.33
	50	1.48	1.81	2.37	2.76	3.32	3.78	4.28
	51	1.46	1.78	2.34	2.73	3.27	3.73	4.23
	52	1.44	1.76	2.31	2.69	3.23	3.69	4.18
	53	1.42	1.74	2.28	2.66	3.19	3.64	4.13
	54	1.41	1.72	2.25	2.63	3.16	3.60	4.08
	55	1.39	1.69	2.22	2.60	3.12	3.56	4.04
	56	1.37	1.67	2.20	2.56	3.08	3.52	3.99
	57	1.35	1.65	2.17	2.53	3.05	3.48	3.95
	58	1.34	1.63	2.14	2.51	3.01	3.44	3.90
	59	1.32	1.61	2.12	2.48	2.98	3.40	3.86
1	60	1.30	1.59	2.10	2.45	2.95	3.37	3.82
2	120	0.78	0.96	1.28	1.51	1.83	2.10	2.40
3	180	0.57	0.71	0.95	1.12	1.36	1.57	1.80
6	360	0.33	0.41	0.56	0.66	0.81	0.93	1.07
12	720	0.19	0.24	0.33	0.39	0.48	0.55	0.63
24	1440	0.11	0.14	0.19	0.22	0.28	0.32	0.37

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Table A-18
500-year storm

		County												County				
		Collin	Dallas	Denton	Ellis	Erath	Grayson	Hood	Hunt	Johnson	Kaufman	Navarro	Palo Pinto	Parker	Rockwall	Somervell	Tarrant	Wise
	e	0.6609	0.6756	0.6819	0.7082	0.7270	0.6637	0.7232	0.6538	0.7118	0.6787	0.7062	0.7520	0.7263	0.6582	0.7198	0.6985	0.7027
	b	70.27	78.09	79.11	99.98	102.47	72.35	100.46	68.71	96.57	82.29	102.16	122.05	101.78	70.46	100.04	87.16	89.48
	d	0	1	1	6	6	1	5	1	5	2	5	8	4	0	6	2	2
Hours	Minutes	Rainfall Intensity (inches per hour)												Rainfall Intensity (inches per hour)				
0.250	15	11.74	12.00	11.94	11.58	11.20	11.49	11.51	11.21	11.45	12.03	12.32	11.55	11.99	11.85	11.18	12.05	12.22
	16	11.25	11.52	11.46	11.20	10.83	11.04	11.11	10.78	11.06	11.57	11.90	11.19	11.55	11.36	10.81	11.58	11.74
	17	10.80	11.08	11.02	10.85	10.49	10.63	10.74	10.38	10.70	11.16	11.51	10.85	11.15	10.92	10.47	11.15	11.30
	18	10.40	10.68	10.62	10.53	10.17	10.25	10.40	10.02	10.36	10.77	11.16	10.53	10.78	10.51	10.16	10.75	10.90
	19	10.04	10.32	10.26	10.23	9.87	9.91	10.09	9.69	10.06	10.42	10.83	10.24	10.44	10.15	9.86	10.39	10.53
	20	9.70	9.99	9.92	9.95	9.59	9.59	9.80	9.39	9.77	10.10	10.52	9.96	10.12	9.81	9.59	10.06	10.20
	21	9.40	9.68	9.61	9.69	9.33	9.30	9.52	9.11	9.50	9.80	10.23	9.70	9.82	9.50	9.33	9.75	9.88
	22	9.11	9.39	9.32	9.44	9.09	9.03	9.27	8.85	9.25	9.52	9.96	9.46	9.55	9.21	9.09	9.47	9.59
	23	8.85	9.12	9.06	9.21	8.86	8.78	9.02	8.60	9.01	9.26	9.71	9.23	9.29	8.95	8.86	9.20	9.32
	24	8.60	8.88	8.81	8.99	8.64	8.54	8.80	8.38	8.79	9.02	9.47	9.01	9.05	8.70	8.65	8.95	9.07
	25	8.37	8.64	8.58	8.78	8.44	8.32	8.59	8.16	8.58	8.79	9.25	8.80	8.82	8.47	8.45	8.72	8.83
	26	8.16	8.43	8.36	8.59	8.25	8.12	8.38	7.96	8.38	8.57	9.04	8.61	8.60	8.25	8.26	8.50	8.61
	27	7.96	8.22	8.15	8.40	8.07	7.92	8.19	7.78	8.19	8.37	8.84	8.42	8.40	8.05	8.08	8.30	8.40
	28	7.77	8.03	7.96	8.23	7.89	7.74	8.01	7.60	8.02	8.18	8.65	8.25	8.21	7.86	7.90	8.10	8.20
	29	7.59	7.85	7.78	8.06	7.73	7.57	7.84	7.43	7.85	8.00	8.47	8.08	8.03	7.68	7.74	7.92	8.01
0.500	30	7.42	7.68	7.61	7.90	7.57	7.41	7.68	7.28	7.69	7.83	8.30	7.92	7.86	7.51	7.59	7.74	7.84
	31	7.26	7.51	7.44	7.75	7.42	7.25	7.52	7.13	7.53	7.67	8.13	7.76	7.69	7.35	7.44	7.58	7.67
	32	7.11	7.36	7.29	7.61	7.28	7.11	7.38	6.99	7.39	7.52	7.98	7.62	7.54	7.20	7.30	7.42	7.51
	33	6.97	7.21	7.14	7.47	7.14	6.97	7.24	6.85	7.25	7.37	7.83	7.48	7.39	7.06	7.16	7.28	7.36
	34	6.83	7.07	7.00	7.33	7.01	6.83	7.10	6.72	7.12	7.23	7.68	7.34	7.25	6.92	7.03	7.13	7.21
	35	6.70	6.94	6.87	7.21	6.89	6.71	6.97	6.60	6.99	7.10	7.55	7.22	7.11	6.79	6.91	7.00	7.08
	36	6.58	6.81	6.74	7.08	6.77	6.59	6.85	6.48	6.87	6.97	7.42	7.09	6.98	6.66	6.79	6.87	6.94
	37	6.46	6.69	6.62	6.97	6.65	6.47	6.73	6.37	6.75	6.85	7.29	6.97	6.86	6.54	6.68	6.75	6.82
	38	6.35	6.57	6.50	6.86	6.54	6.36	6.62	6.26	6.64	6.73	7.17	6.86	6.74	6.43	6.57	6.63	6.70
	39	6.24	6.46	6.39	6.75	6.44	6.25	6.51	6.16	6.53	6.62	7.06	6.75	6.62	6.32	6.46	6.51	6.58
	40	6.14	6.35	6.29	6.64	6.34	6.15	6.40	6.06	6.43	6.51	6.95	6.64	6.52	6.22	6.36	6.41	6.47
	41	6.04	6.25	6.18	6.54	6.24	6.05	6.30	5.97	6.33	6.41	6.84	6.54	6.41	6.12	6.26	6.30	6.37
	42	5.94	6.15	6.09	6.45	6.14	5.96	6.21	5.88	6.23	6.31	6.74	6.44	6.31	6.02	6.17	6.20	6.26
	43	5.85	6.06	5.99	6.35	6.05	5.87	6.11	5.79	6.14	6.21	6.64	6.35	6.21	5.93	6.08	6.10	6.17
	44	5.76	5.97	5.90	6.26	5.96	5.78	6.02	5.70	6.05	6.12	6.54	6.25	6.12	5.84	5.99	6.01	6.07
0.750	45	5.68	5.88	5.81	6.17	5.88	5.70	5.93	5.62	5.96	6.03	6.45	6.17	6.03	5.75	5.90	5.92	5.98
	46	5.60	5.79	5.73	6.09	5.79	5.62	5.85	5.54	5.88	5.95	6.36	6.08	5.94	5.67	5.82	5.83	5.89
	47	5.52	5.71	5.65	6.01	5.72	5.54	5.77	5.47	5.80	5.86	6.27	6.00	5.85	5.59	5.74	5.75	5.81
	48	5.44	5.63	5.57	5.93	5.64	5.47	5.69	5.39	5.72	5.78	6.19	5.92	5.77	5.51	5.67	5.67	5.73
	49	5.37	5.56	5.49	5.85	5.56	5.39	5.61	5.32	5.65	5.71	6.11	5.84	5.69	5.44	5.59	5.59	5.65
	50	5.30	5.48	5.42	5.78	5.49	5.32	5.54	5.26	5.57	5.63	6.03	5.76	5.61	5.37	5.52	5.52	5.57
	51	5.23	5.41	5.35	5.71	5.42	5.25	5.47	5.19	5.50	5.56	5.95	5.69	5.54	5.30	5.45	5.44	5.50
	52	5.16	5.34	5.28	5.64	5.35	5.19	5.40	5.12	5.43	5.49	5.88	5.62	5.47	5.23	5.38	5.37	5.42
	53	5.10	5.28	5.21	5.57	5.29	5.12	5.33	5.06	5.37	5.42	5.81	5.55	5.40	5.17	5.32	5.31	5.36
	54	5.03	5.21	5.15	5.50	5.22	5.06	5.26	5.00	5.30	5.36	5.74	5.48	5.33	5.10	5.25	5.24	5.29
	55	4.97	5.15	5.08	5.44	5.16	5.00	5.20	4.94	5.24	5.29	5.67	5.41	5.26	5.04	5.19	5.17	5.22
	56	4.91	5.09	5.02	5.38	5.10	4.94	5.14	4.89	5.18	5.23	5.60	5.35	5.20	4.98	5.13	5.11	5.16
	57	4.86	5.03	4.96	5.32	5.04	4.89	5.08	4.83	5.12	5.17	5.54	5.29	5.14	4.92	5.07	5.05	5.10
	58	4.80	4.97	4.90	5.26	4.98	4.83	5.02	4.78	5.06	5.11	5.48	5.23	5.08	4.87	5.01	4.99	5.04
	59	4.75	4.91	4.85	5.20	4.93	4.78	4.96	4.73	5.00	5.05	5.42	5.17	5.02	4.81	4.96	4.94	4.98
1	60	4.69	4.86	4.79	5.14	4.87	4.73	4.91	4.67	4.95	5.00	5.36	5.11	4.96	4.76	4.90	4.88	4.92
2	120	2.97	3.06	3.01	3.25	3.05	3.00	3.06	2.99	3.11	3.16	3.38	3.18	3.07	3.02	3.08	3.04	3.06
3	180	2.27	2.33	2.28	2.47	2.29	2.30	2.30	2.30	2.35	2.41	2.56	2.38	2.30	2.31	2.33	2.30	2.31
6	360	1.44	1.46	1.43	1.53	1.40	1.45	1.41	1.46	1.45	1.51	1.58	1.44	1.40	1.46	1.43	1.42	1.42
12	720	0.91	0.92	0.89	0.94	0.85	0.92	0.86	0.93	0.89	0.94	0.98	0.86	0.85	0.93	0.87	0.88	0.88
24	1440	0.57	0.57	0.55	0.58	0.52	0.58	0.52	0.59	0.54	0.59	0.60	0.51	0.52	0.59	0.53	0.54	0.54

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APPENDIX

B

HYDROLOGIC SOILS DATA

Electronic Soil Maps

Electronic soils data in the Soil Survey Geographic (SSURGO) Database can be obtained free of charge from the National Resource Conservation Service (NRCS) at http://soildatamart.nrcs.usda.gov. The data is downloadable by county and includes extensive soil information, including hydrologic soil groups. The data is intended to be imported into a geographic information system (GIS) to allow for site-specific soil analysis of soil characteristics for storm design. All soil survey results can also be accessed online at http://websoilsurvey.nrcs.usda.gov/app/. Maps can be created and printed from this site without the use of a GIS.

B.1 Online Web Soil Survey

Following is a procedure for using the NRCS online Web Soil Survey.

1. Go to http://websoilsurvey.nrcs.usda.gov/app/
2. Click **Start WSS**
3. Define your Area of Interest by drawing a box around your site location.
4. Click the **Soil Map** tab
5. Click **Save or Print** in the upper right hand corner. A pdf will open in a new window that you can either print or save. It will show the area of interest along with a legend and the appropriate map units.

B.2 Downloading Soil Surveys

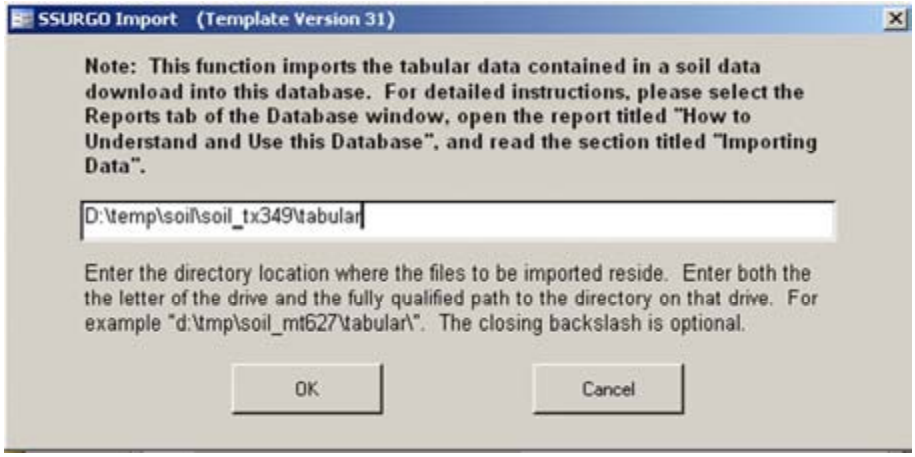
Following is a procedure for downloading data from the NRCS web site and importing it into ArcGIS.

<u>Downloading SSURGO Soil Data into ArcInfo 9.x</u>

1. Go to http://soildatamart.nrcs.usda.gov
2. Click **Select State**
3. Select State (**Texas)**
4. Select **County** of interest
5. Click **Select Survey Area**
6. Click **Download Data**
7. Enter your e-mail address in the provided form space
8. Click **Submit Request**
9. You will receive an immediate message acknowledging your request and a follow-up e-mail once your request has been processed.
10. The file(s) will be provided in compressed ZIP format, requiring the use of WinZip to extract.
11. Extract the files to a destination directory of your choice. The extracted files contain a spatial sub-folder, a tabular sub-folder, and a zip file containing the SSURGO MS Access template file.

<u>Importing raw tabular soil data into Microsoft Access</u>

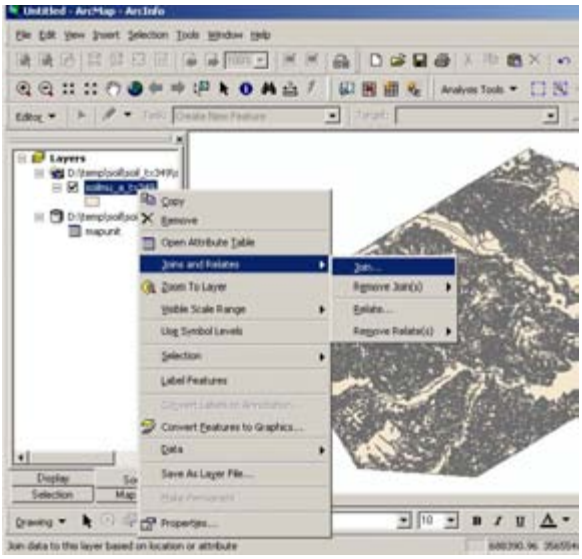
1. Extract the soildb_US_2002.zip file into the same destination folder by using the “extract to here” command in WinZip. This will extract the template database.
2. Open the template database and input the path name to the tabular data. This will build the SSURGO database and allow the creation of reports and queries.

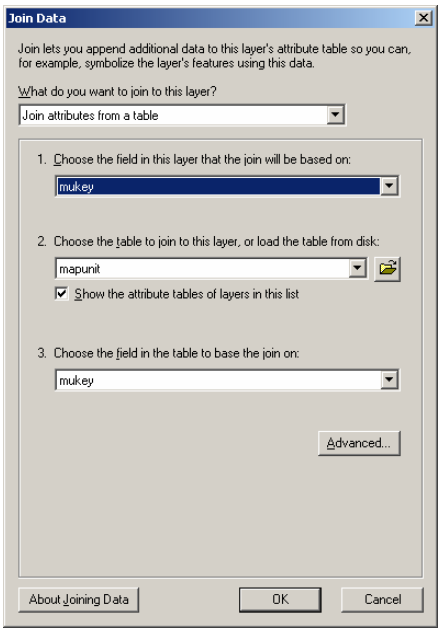


3. Once the data is imported into the database, a report can be run. With the soil reports dialog box up, press the **Select All** option and generate the report. Note: Regardless of what report you wish to run, all reports are simultaneously created. The selected report is displayed on the screen.
4. All the reports are now complete, and the tables can now be added directly into ArcGIS.

<u>Joining tables to shapefiles in ArcGIS</u>

1. Open **ArcGIS** and add the **Soils** shapefile.
2. Add the "**mapunit**" report to ArcGIS by navigating directly to the MS Access database and opening it (via the add data dialog box). Note: mapunit is only a commonly used example, containing full soil names and prime farmland information.
3. Now that the table is added to the Table of Contents, it is ready to be joined to the existing soils shapefile.
4. Right click on the **soils** shapefile and select **join.**





5. Under the **Join Data** dialog box, select the **mukey** field in Dropdown Box 1 and select **mapunit** in Dropdown Box 2.
6. Now that your shapefile is joined with the appropriate information, the next step is to export the shapefile into a new shapefile with the joins saved permanently. Right click on the **soils** shapefile and choose **Data > Export** and **Save** your file.

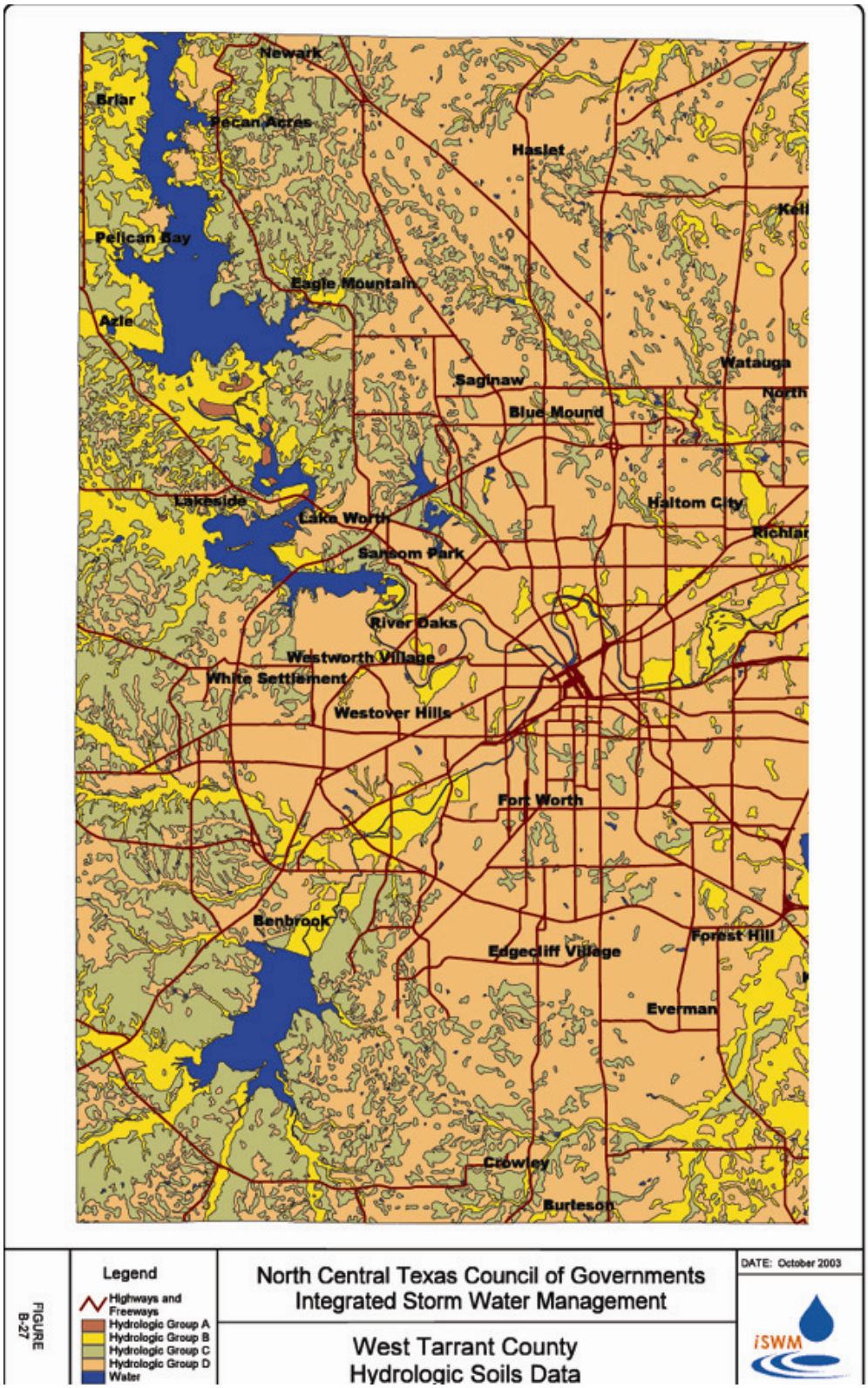


Figure B-1. Example SSURGO Soil Map – West Tarrant County

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APPENDIX

C

FEDERAL, STATE & REGIONAL INITIATIVES

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C.1 Overview

As a result of the need to address the potential negative impacts of development and storm water runoff, numerous federal, state, and regional programs and regulations have been created to deal with the problems of urban runoff and nonpoint source pollution. A storm water management program is best implemented at a local level, where the local community directly influences land use and development related decisions. Federal and state legislation greatly influences and supports local governments in their efforts to manage storm water runoff in their communities.

It is the intent of iSWM to complement the federal, state, and regional initiatives in an integrated partnership approach to storm water management.

The purpose of this section is to provide a brief overview of many of the regional, state, and federal laws, regulations, and programs that are required of local government entities in the North Central Texas region that may impact local storm water management programs and activities. The initiatives are summarized in Table C.2-1 as a general guide for developers and communities indicating areas where federal, state, or regional initiatives may affect their project.

As this is not intended to be a detailed analysis of each requirement, it would be advisable to obtain a copy of the specific administrative rules for each program from the appropriate regulatory agency.

C.2 Federal Initiatives

C.2.1 National Flood Insurance Program

Established under the National Flood Insurance Act of 1968 and broadened with the passage of the Flood Disaster Protection Act of 1973, the National Flood Insurance Program (NFIP) provides federally supported flood insurance to residents in communities that voluntarily adopt and enforce regulations to reduce future flood damage. As part of the program, the Federal Emergency Management Agency (FEMA) defines minimum standards for floodplain development that the local communities must adopt to be eligible for program benefits. New construction and substantial improvements must be built at or above the base flood elevation, which is the computed elevation of the “100 year flood”. Also, new development that would result in an increase in flood heights is prohibited in the defined floodways. More information on the NFIP and floodplain management in general is available from the FEMA – Region VI office in Denton, TX.

Contact Agency	Phone	Address
Federal Emergency Management Agency (FEMA) Region VI	940-898-5399	800 N. Loop 288 Denton, TX 76209

Website: http://www.fema.gov/regions/vi/index.shtm

C.2.2 U.S. Army Corps of Engineers Regulatory Program

The U.S. Army Corps of Engineers (USACE) Regulatory Program implements Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act of 1899, through regulations that serve to protect the Nation’s valuable aquatic resources.

Under Section 404 of the Clean Water Act, permits are required for the discharge of dredged or fill material into waters of the United States (U.S.). Waters of the U.S. include all surface waters, such as coastal and navigable inland waters, lakes, rivers, streams, and their tributaries; interstate waters and their tributaries; wetlands adjacent to the above (e.g. swamps, marshes, bogs, or other land areas); and isolated wetlands and lakes, intermittent streams, and other waters where degradation could affect

interstate commerce. The TCEQ is responsible for conducting water quality certification reviews for 404 permits in Texas. The certification reviews are required by Section 401 of the Clean Water Act. The goal of these certification reviews is to determine whether a proposed discharge will comply with state water quality standards. The Railroad Commission of Texas is the certifying agency in Texas for projects associated with oil and gas.

Under Section 10 of the Rivers and Harbors Act of 1899, the Corps regulates all work and structures in or affecting the course, condition, or capacity of navigable waters of the U.S. Navigable waters of the U.S. are those waters that are subject to the ebb and flow of the tide shoreward to the mean high water mark and/or are presently used, have been used in the past, or may be susceptible to use in the transport of interstate or foreign commerce.

Contact Agency	Phone	Address
Texas Commission on Environmental Quality	512-239-0400	MC 174 P.O. Box 13087, Austin, TX 78711-3087
US Army Corps of Engineers, Fort Worth District (for North Area of State)	817-886-1731	CESWF-PER-R 819 Taylor St P.O. Box 17300 Fort Worth, TX 76102-0300

Website: www.tceq.state.tx.us and http://www.swf.usace.army.mil

C.2.3 National Environmental Policy Act, Council on Environmental Quality

Congress established the Council on Environmental Quality within the Executive Office of the President as part of the National Environmental Policy Act of 1969 (NEPA). NEPA establishes the Federal government's environmental policy to help public officials make decisions based on an understanding of the environmental consequences of their actions, and to take actions that protect, restore, and enhance the environment. To accomplish this, NEPA requires Federal agencies to either prepare or have prepared written assessments of statements that describe the:

- Affected environment and environmental consequences of a proposed project;
- Reasonable or practical alternatives to the proposed project; and
- Any mitigation measures necessary to avoid or minimize adverse environmental effects.

If a project is either directly or indirectly funded by a federal source, then the agency providing funds to the project owner is responsible for seeing that the appropriate level of environmental assessment is performed.

C.2.4 Wildlife and Habitat

The mission of the U.S. Fish and Wildlife Service (USFWS) is to conserve, protect, and enhance fish and wildlife, and their habitats for the continuing benefit of the American people. USFWS activities include, but are not limited to enforcing the federal Endangered Species Act (ESA); acquiring wetlands, fishery habitats, and other lands for restoration and preservation; insuring compliance with the National Environmental Policy Act (NEPA); managing National Wildlife Refuges and National Fish Hatcheries; and reviewing and commenting on all water resource projects.

Table C.2-1 Federal, State and Regional Initiative Indicators

Indicator Questions	Potentially Applicable Initiative(s)
Are **Federal funds** to be used? Is a **Federal Action Permit** required?	**National Environmental Policy Act, Council on Environmental Quality** (Section C.2.3) **Federal Agency with Action**
Are **structures** or **manmade features** to be disturbed?	**Texas Historical Commission** (Section C.3.11)
Is this a **construction** project with land disturbing activity?	**TPDES Storm Water Permits for Construction Activities** (Section C.3.2)
Is there new **landscaping** and/or **erosion potential**?	**Local Ordinances**
Will storm water **discharge to an MS4**?	**TPDES MS4 Storm Water Permit Program (Phase I and II)** (Section C.3.1)
Is this an **industrial** facility?	**TPDES Industrial Storm Water Permit Program** (Section C.3.3)
Is there a **wastewater** discharge?	**TPDES Wastewater Discharge Permit Program** (Section C.3.4)
Is there a nearby **public water supply**?	**Source Water Assessment Program** (Section C.3.8)
Are there nearby **groundwater** usages?	**Texas Groundwater Protection Committee Local Groundwater District** (Section C.3.9)
Are there adjacent or nearby **surface water features** or **wetlands** that may be affected?	**U.S. Army Corps of Engineers Regulatory Program** (Section C.2.2) **Texas Railroad Commission** (Section C.2.2) **Water Quality Management Plan – TCEQ** (Section C.3.5) **Surface Water Quality Standards – TCEQ** (Section C.3.6) **Total Maximum Daily Load Program – TCEQ** (Section C.3.7) **Texas General Land Office** (Section C.3.12)
Will there be **dredging**?	**U.S. Army Corps of Engineers Regulatory Program** (Section C.2.2) **Sand, Gravel, Shell, and Marl Protection – TCEQ** (Section C.3.13) **Texas Parks and Wildlife Department** (Section C.3.14)
Will a **dam** be constructed or modified?	**U.S. Army Corps of Engineers Regulatory Program** (Section C.2.2) **Dam Safety Program – TCEQ** (Section C.3.10)
Is the project in a **floodplain**?	**National Flood Insurance Program** (Section C.2.1) **Hazard Mitigation Action Planning** (Section C.4.2)
Is the project in the Regulatory Zone of the **Trinity River Corridor**?	**Trinity River Corridor Development Certificate** (Section C.4.1) **SAFE CLEAN & GREEN** (Section C.4.3)
Are there potential impacts on **fish, wildlife,** or **habitats**?	**Texas Parks and Wildlife Department** (Section C.3.14) **U.S. Fish and Wildlife Service** (Section C.2.4)

C.3 State of Texas Initiatives

C.3.1 TPDES MS4 Storm Water Permit Program (Phase I and II)

The National Pollutant Discharge Elimination System (NPDES) permit system was originally established by the Clean Water Act of 1972 to control wastewater discharges from various industries and wastewater treatment plants known as "point" sources. Congress amended the Clean Water Act with the Water Quality Act of 1987 to expand the NPDES permit program to address "nonpoint" source pollution through schedules for permitting municipal storm water discharges. The Municipal Separate Storm Sewer System (MS4) storm water discharge permit establishes requirements for municipalities to minimize pollutants in storm water runoff to the "maximum extent practicable."

In Texas, the Texas Commission on Environmental Quality (TCEQ) has authority to administer the NPDES program (TPDES – Texas Pollutant Discharge Elimination System). Under the TCEQ's Municipal Separate Storm Sewer System (MS4) permit program, local governments and public entities in regulated areas are required to establish and implement a comprehensive Storm Water Management Program (SWMP) to control potential pollutants in storm water runoff discharging to Waters in the State to the maximum extent practicable and to eliminate non-storm water discharges entering the storm water system.

Contact Agency	Phone	Address
TCEQ – Storm Water and General Permits Team	512-239-4527	MC 148 P.O. Box 13087, Austin, TX 789711-3087

Website: www.tceq.state.tx.us and www.dfwstormwater.com

This is accomplished by a municipal program which includes the implementation of structural and non-structural storm water measures to protect storm water quality. These measures are called best management practices (BMPs). BMPs include public education and public involvement activities, ordinance or other enforcement mechanisms, inspections and enforcement, employee training, and structural and non-structural controls. Storm water management ordinances, erosion and sediment control ordinances, development regulations, and other local regulations provide enforcement authority and criteria for controlling significant components of the Storm Water Management Program.

Since 1993, the Phase I permit requirements have applied in Texas to large and medium municipal separate storm sewer systems (defined by a population greater than 250,000 and population between 100,000 and 250,000, respectively, based on the 1990 census, or those areas contributing to water quality violations). In Texas, 22 entities are in the Phase I program, including the following seven local governments in North Central Texas:

- City of Dallas
- City of Fort Worth
- City of Plano
- City of Arlington
- City of Garland
- City of Irving
- City of Mesquite

Federal regulations were issued in 1999 to extend the NPDES MS4 permit program to smaller (Phase II) communities. In Texas, the TCEQ administers this permit program. The Phase II rules take a slightly different approach than Phase I rules for the implementation of local storm water management programs. Unlike Phase I communities that were required to develop detailed individual permits, each Phase II community that is the operator of a municipal storm sewer system will be required to develop a storm water management program (SWMP) to protect water quality. The SWMP will be required to either meet the conditions of a general permit or an individual permit authorized by the TCEQ.

Under the TCEQ Phase II MS4 general permit (Proposed General Permit No. TXR040000), regulated entities are required to develop and submit to the TCEQ a plan for their SWMP within 180 days from the date the permit is issued. Additional communities may be added to the Phase II program if the TCEQ determines that storm water in the community is adversely impacting the quality of receiving waters such as streams, lakes, and estuaries. Communities added to the program by the TCEQ will have 180 days from the date of notification to develop and submit a plan for their SWMP.

The U.S. EPA and TCEQ established six elements, termed "minimum control measures," that each Phase II SWMP must address. The TCEQ has added an optional seventh minimum control measure. The control measures are listed below:

1. Public Education and Outreach
2. Public Participation / Involvement
3. Illicit Discharge Detection and Elimination
4. Construction Site Runoff Control
5. Post-Construction Runoff Control
6. Pollution Prevention / Good Housekeeping
7. Municipal Construction Activity

The seventh optional minimum control measure allows regulated communities to include municipal construction activity in the MS4 general permit instead of having to obtain a permit for each construction site over five acres. The goal of this option is to reduce paperwork for regulated communities with numerous construction projects.

Each Phase II MS4 community is required to develop and submit a plan for their SWMP to the TCEQ. In the plan, the regulated community must identify its selection of BMPs for each required minimum control measure. Additionally, measurable goals to track the implementation progress and success of each BMP must be identified, and a timeline is required for the implementation of the BMPs over the five-year permit term.

For the past decade, NCTCOG on a fee-for-service basis has been providing assistance to the seven Phase I MS4 communities and TxDOT. This program was expanded in the late 1990's to all potential Phase II MS4 communities in the region as well. An overview of this program can be found in Section C.4.4.

The iSWM manual will provide tremendous assistance to the MS4 communities in the development and implementation of their SWMPs. The iSWM guidelines will specifically apply to aspects of several of the minimum control measures, thus providing the MS4 communities with the framework to accomplish these measures.

C.3.2 TPDES Storm Water Permits for Construction Activities

The TPDES storm water permit for construction activities is directed toward controlling the quality of storm water runoff from construction activities. The permit requires the development of a construction Storm Water Pollution Prevention Plan (SWPPP) that emphasizes the application of BMPs to protect storm water quality from erosion and sedimentation processes, as well as construction materials and wastes, during the construction phase of development. Guidance for practices related to storm water quality protection for construction site activities can be found in the iSWM Design Manual for Construction. In Texas, this program which became effective March 5, 2003, is managed by TCEQ.

Operators of construction sites five acres or greater are required to obtain storm water permits from the TCEQ by developing a SWPPP and filing a Notice of Intent (NOI) 48 hours prior to initiating construction activities.

Construction sites one to five acres in size require a SWPPP to be developed, but an NOI is not required to be submitted to the TCEQ. Construction sites that are part of a larger common plan of development, such as a subdivision, that is collectively greater than one or five acres, must be evaluated according to the size of the larger common plan of development when considering permit requirements. For example,

a person constructing on a ¼-acre site located within a 10-acre subdivision under construction would be required to comply with the permit requirements for sites disturbing 5 acres or more.

If storm water runoff from the construction site enters a municipal storm sewer system, the construction site operator and/or owner is also required to notify the MS4 operator about the construction activity. For large construction sites, submitting a copy of the NOI to the MS4 operator is required, and for small construction sites, a copy of the Construction Site Notice is required. Local ordinances should be checked for additional requirements. For example, in some communities the SWPPP must also be submitted to the MS4 operator.

Construction site operators on sites with an NOI are further required to submit a Notice of Termination (NOT) to TCEQ when final stabilization has been achieved on all portions of the site under their control. Refer to TPDES General Permit TXR150000 for specific permit requirements.

C.3.3 Industrial TPDES Storm Water Permit Program

The TPDES program also requires that the discharge of storm water from certain types of industrial facilities be regulated under a permit program. Industrial storm water is defined as that discharged from any conveyance which is used for collecting and conveying storm water and which is directly related to manufacturing, processing, or materials storage areas. Discharge of storm water from regulated industrial facilities in Texas is managed by the TCEQ under a single general permit issued in 2001.

Currently, the following thirty categories of industrial facilities identified in the Multi-Sector General Permit are required to have a TPDES permit for their storm water discharge:

- Timber Products
- Paper and Allied Products
- Chemical and Allied Products
- Asphalt Paving and Roofing Materials and Lubricants
- Glass, Clay, Cement, Concrete, and Gypsum Products
- Primary Metals
- Metal Mining (Ore Mining and Dressing)
- Coal Mines and Coal Mining Related Facilities
- Oil and Gas Extraction
- Mineral Mining and Dressing
- Hazardous Waste Storage Facilities
- Landfills and Land Application Sites
- Automobile Salvage Yards
- Scrap Recycling Facilities
- Steam Electric Generating Facilities
- Land Transportation and Warehousing
- Water Transportation
- Ship and Boat Building or Repairing Yards
- Air Transportation
- Treatment Works
- Food and Kindred Products
- Textile Mills, Apparel, and Other Fabric Product Manufacturing, Leather and Leather Products
- Furniture and Fixtures
- Printing and Publishing
- Rubber, Miscellaneous Plastic Products, and Miscellaneous Manufacturing Industries
- Leather Tanning and Finishing
- Fabricated Metal Products
- Transportation Equipment, Industrial or Commercial Machinery
- Electronic, Electrical, Photographic, and Optical Goods
- Miscellaneous Industrial Activities

Regulated industrial facilities are required to develop a storm water pollution prevention plan (SWPPP) and submit a Notice of Intent (NOI) for permit coverage to the TCEQ. Components of the SWPPP include

identification and elimination of potential sources of storm water contamination, storm water monitoring at each storm water outfall, employee training, and other storm water protection activities. New industrial facilities are required to submit an NOI 48 hours prior to conducting any new activity.

C.3.4 TPDES Wastewater Discharge Permit Program

Municipalities, communities, and industrial facilities are required to obtain an NPDES permit to discharge wastewater to waters of the U.S. These point source discharges have been regulated by the EPA for more than 30 years. The TCEQ administers the wastewater discharge permit program in Texas. Permit effluent limitations are developed based on the quality of the receiving water. More stringent limitations are placed on discharges to waters listed on the 303(d) list of impaired waters. As part of the permitting process, applicants are required to characterize the quality of the proposed effluent, evaluate the conditions of the receiving water, identify nearby water supply intakes, and conduct other activities to document the continued protection of water quality.

Contact Agency	Phone	Address
TCEQ – Wastewater Permitting Section	512-239-4433	MC 148 P.O. Box 13087 Austin, TX 789711-3087

Website: www.tceq.state.tx.us and www.tnrcc.state.tx.us/water/quality/tmdl/index.html

C.3.5 Water Quality Management Plan

The Water Quality Management Plan (WQMP) for Texas is developed and updated by the TCEQ as part of the requirements of the Clean Water Act. The plan provides long-range planning and technical data for water quality management activities required under the Texas Water Code and the Clean Water Act. Projected effluent limitations for dischargers are based on the recommendations in the WQMP. The Texas State Soil and Water Conservation Board (TSSWCB) is responsible for addressing water quality issues from nonpoint source pollution from agricultural and silvicultural activities. All new and amended permits are reviewed by the TCEQ for conformance with the recommendations of the WQMP. The plan is updated on a quarterly basis by the State.

For North Central Texas, NCTCOG is the designated water quality management planning agency. Under guidance of its Water Resources Council, NCTCOG annually updates the plan and submits amendments to the TCEQ for certification. The annual water quality management plan is part of the SEE Safe Clean and Green initiative outlined in Section C.4.3.

Contact Agency	Phone	Address
TCEQ – Office of Environmental Policy, Analysis, and Assessment	817-239-4900	MC 147 P.O. Box 13087, Austin, TX 789711-3087
TSSWCB – Technical Assistance	254-773-2250	Temple, TX 76501
NCTCOG – Regional Plan	817-695-9210	P.O. Box 5888 Arlington, TX 76005-5888

Website: www.tceq.state.tx.us and www.dfwstormwater.com

C.3.6 Surface Water Quality Standards

The TCEQ sets surface water quality standards in an effort to maintain the quality of water in the state consistent with public health and enjoyment, protection of aquatic life, operation of existing industries, and economic development of the state, as well as to encourage and promote development and use of regional and area-wide wastewater collection, treatment, and disposal systems. Standards are implemented primarily through effluent limits established in permits that allow discharges of wastewater into the surface waters of the state. Currently, the State of Texas has no numerical criteria for nutrients in the Texas Surface Water Quality Standards (TSWQS). The EPA is requiring states to develop nutrient

criteria. The TCEQ is currently developing and evaluating criteria to address nutrients and eutrophication.

Contact Agency	Phone	Address
TCEQ – Water Quality Assessment Section	817-239-4576	MC 150 P.O. Box 13087, Austin, TX 789711-3087

Website: www.tceq.state.tx.us

C.3.7 Total Maximum Daily Load (TMDL) Program

Under Section 303(d) of the Clean Water Act, the State of Texas is required to develop a list of impaired waters that do not meet water quality standards. The TCEQ must then establish priority rankings for waters on the list and develop Total Maximum Daily Loads (TMDLs) for listed waters. The TMDL specifies the maximum amount of a specific pollutant of concern that a designated segment of a water body can receive and still meet water quality standards. The TMDL also allocates pollutant loadings among point and nonpoint pollutant sources, including storm water runoff. If a TMDL has been established for a water body, the impact of new point and nonpoint sources must be assessed prior to TCEQ approval of new or amended discharge or storm water permits.

For each pollutant identified, a TMDL implementation plan must be developed. The implementation plans must identify the sources of the pollutant and provide a list of actions or management measures needed to reduce the pollutant, a schedule for implementing controls or measures, milestones for implementation, and a monitoring program to measure progress. Controls and management measures need to be in place five years after the plan is developed.

The TMDL program has a broad impact on local storm water management programs because nonpoint sources of pollutants must be addressed at the local level. Currently, the TMDL requirements have been withdrawn and are under evaluation by EPA to determine whether and how the TMDL program can best be implemented to achieve the goals of the Clean Water Act. However, there are TMDLs that have already been approved for segments of the Trinity River Watershed.

Contact Agency	Phone	Address
U.S. EPA – Region 6	214- 665-6444	1445 Ross Avenue, Suite 1200 Dallas, TX 75202 (214) 665-6444
TCEQ – TMDL Section	817-239-4300	MC 150 P.O. Box 13087, Austin, TX 789711-3087

Website: http://www.epa.gov/region6/index.htm and http://www.tceq.state.tx.us

C.3.8 Source Water Assessment Program (SWAP)

The 1996 amendments to the Federal Safe Drinking Water Act brought about a new approach for ensuring clean and safe drinking water served by public water supplies known as the Source Water Assessment Program. The U.S. EPA is advocating prevention as an important tool in the protection of public drinking water sources from contamination. In order to implement source protection, an assessment of potential pollutant sources in water supply watersheds must be conducted. The goals of this assessment will be reached through implementation of a four-step method which includes watershed delineation, inventory of potential pollutant sources within the watershed, analysis of susceptibility of a water intake to the pollutant sources, and communication of this information to the public.

As many pollutants can enter waterways and reservoirs through storm water drainage systems, the SWAP efforts will provide an informational resource to local storm water pollution prevention and mitigation programs. Future water supply protection efforts to control the identified pollution sources should be coordinated with and included as part of a local storm water program.

The TCEQ's Source Water Assessment Program consists of source water protection, source water assessments, and vulnerability assessments. The Source Water Protection (SWP) program is a voluntary program that public water supply systems participate in to prevent pollution from entering surface or ground water supplies. It was created by the 1996 Amendments to the Safe Drinking Water Act and is an expansion of the Wellhead Protection (WHP) program, which protects public water supplies that use groundwater. Source water assessments and vulnerability assessments are conducted to evaluate a public water supply systems susceptibility to a wide range of contaminants.

Contact Agency	Phone	Address
TCEQ – Public Drinking Water Section	512-239-4782	MC 155 P.O. Box 13087, Austin, TX 789711-3087

Website: www.tceq.state.tx.us

C.3.9 Texas Groundwater Protection Committee

The Texas legislature established the Texas Groundwater Protection Committee (TGPC) in 1989 with a specific charge to "*. . . develop and update a comprehensive groundwater protection strategy for the state that provides guidelines for the prevention of contamination and for the conservation of groundwater and that provides for the coordination of the groundwater protection activities of the agencies represented on the committee...* (Water Code Section 26.405(2))*."* The TGPC first established a groundwater protection strategy in 1988 and recently updated the strategy (February 2003). Implementation of the strategy is primarily through local groundwater districts, but may also be conducted through River Authorities and Regional Water Planning Groups, municipalities by ordinances, or county on-site treatment regulatory programs.

Contact Agency	Phone	Address
TCEQ – Texas Groundwater Protection Committee	512-239-4506	MC 197 P.O. Box 13087, Austin, TX 789711-3087

Website: www.tgpc.state.tx.us

C.3.10 Dams and Reservoirs in Texas

The Texas Commission on Environmental Quality (TCEQ) regulates the construction of dams in Texas that are six feet or more in height as per the Texas Administrative Code (Title 30, Part 1, Chapter 299 – Dams and Reservoirs). Approval from the TCEQ of plans and specifications is required for construction of a dam. The TCEQ also has the authority to inspect existing dams, and if necessary, require unsafe dams to be upgraded or removed. The Dam Safety Program is administered under the Field Operations Division of the Office of Compliance and Enforcement. More information on dams and reservoirs can be found in Appendix H and on-line at www.tnrcc.state.tx.us/enforcement/dam_safety/intro2.html.

The U.S. Army Corps of Engineers Regulatory Program implements Section 9 of the Rivers and Harbors Act of 1899. Under Section 9, permits are required for the construction of dams and dikes in navigable waters of the U.S. See www.swf.usace.army.mil/pubdata/environ/regulatory/jurisdiction/navlist.pdf for a list of navigable waters of the U.S. for the Fort Worth, Tulsa, and Albuquerque Districts within the State of Texas.

C.3.11 Texas Historical Commission

The Texas Historical Commission (THC) is the state agency for historic preservation. THC staff consults with citizens and organizations to preserve Texas' architectural, archeological, and cultural landmarks. The agency maintains the *Texas Historic Atlas*, a directory of historic site records, including data on Texas Historical Markers and National Register of Historic Places in Texas. Projects that include the

disturbance of existing features such as buildings or other manmade structures should include a search of the THC database to screen for potential historic significance. Projects that include ground disturbance, such as new construction or pipeline installation, should include an assessment of the potential for disturbance of archeological or other culturally significant sites. The contact information for the THC is below.

Contact Agency	Phone	Address
Texas Historical Commission	512-463-6100	P.O. Box 12276 Austin, TX 78711-2276

Website: http://www.thc.state.tx.us

C.3.12 Texas General Land Office

A General Land Office (GLO) easement may be required if the project crosses or disturbs any state-owned streambeds. The ownership of any stream is determined on a case-by-case basis by the GLO.

Contact Agency	Phone	Address
Texas General Land Office	1-800-998-4456 *or* 512-463-5001	1700 N. Congress Ave. Suite 840 Austin, TX 78701-1495

Website: http://www.glo.state.tx.us

C.3.13 Sand, Gravel, Shell, and Marl Protection

The Texas Parks and Wildlife Department (TPWD) requires a permit for a project if it "disturbs or takes" streambed materials from a streambed claimed by the State. If the stream is perennial (flows most of the time), or is more than 30 feet wide between the banks (even if it is dry most of the time), the State might claim the bed and the sand and gravel in it as State-owned.

An application must be filed with the TPWD including information on the size of the stream; the nature of the banks and the bed of the stream; the amount of material requested; the adjacent landowners; and the probable effects on the stream and its other users. The TPWD evaluates the probable impacts of the activity, and grants a permit if no significant damage is anticipated. Payment is required for any material removed, in addition to the application fee. A hearing may be required before deciding whether to issue the permit.

C.3.14 Wildlife and Habitat

The Texas Parks and Wildlife Department (TPWD) Resource Protection Division is charged with protecting the state's fish, wildlife, plant, and mineral resources from degradation or depletion. The TPWD investigates environmental contamination events that may result in impacts to fish or wildlife. The agency also provides information and recommendations to other government agencies and participates in administrative and judicial proceedings concerning pollution incidents, development projects, and other actions that may affect fish and wildlife. The Department investigates fish kills, and when responsibility for fish kills can be traced, the agency recovers from the responsible party the economic value of the fish and other lost aquatic life, which by law are state properties. As a state natural resource trustee, the TPWD assesses damages to fish and wildlife resources from oil and hazardous chemical releases and seeks restoration of these resources from the responsible party.

C.4 Regional Initiatives

The North Central Texas Council of Governments (NCTCOG) is a voluntary association of, by, and for local governments, and was established to assist local governments in planning for common needs, cooperating for mutual benefit, and coordinating for sound regional development. NCTCOG's purpose is to strengthen both the individual and collective power of local governments and to help them recognize regional opportunities, eliminate unnecessary duplication, and make joint decisions.

What is SEE? It's an acronym for Sustainable Environmental Excellence. It was first used as the title of a regional environmental conference hosted by the North Central Texas Council of Governments (NCTCOG) in August 2001. That conference helped define the most pressing issues and challenges facing the region and reemphasized a cooperative approach to solve them.

Following months of work with local governments and others, NCTCOG's Executive Board approved two plans—SEE Less Trash and SEE Safe Clean & Green Regional Environmental Corridors—as well as the mission and 10 principles of the Center of Development Excellence in June 2002. Local leaders and NCTCOG are now implementing elements of these plans and principles through numerous projects. There is also an initiative to SEE Clean Air, which is currently focused on the review of the region's "State Implementation Plan" for air quality.

NCTCOG's Department of Environment and Development supports numerous committees composed of local government officials and staff, representatives of business, industry, and nonprofit organization, as well as private citizens, to address these issues. These volunteers make possible a variety of projects and programs dealing with municipal solid waste, public works, building codes, storm water management, flooding, water quality, air quality, recycling, trails, development, and more.

The principles of SEE – moving forward in a way that benefits all North Central Texas today and into the 21st Century – now help guide the efforts of NCTCOG's Department of Environment and Development, and ultimately the region, as we collaborate to plan for this growing and changing area.

C.4.1 Trinity River Corridor Development Certificate

The Corridor Development Certificate (CDC) process aims to stabilize flood risk along the Trinity River. The CDC process does not prohibit floodplain development, but ensures that any development that does occur in the floodplain will not raise flood water levels, reduce flood storage capacity, or increase erosive water velocities.

Under the CDC process, a CDC permit is required to develop land within a specific area of the Trinity floodplain called the Regulatory Zone, which is similar to the 100-year floodplain. An application for a CDC Certificate must be filed with the appropriate local government floodplain administrator. The CDC permit application is forwarded to each of the local governments in the Trinity River Corridor for review and comment.

The US Army Corps of Engineers will also provide a Preliminary Technical Review of hydrologic and hydraulic information contained in the CDC application. Each individual local government, however, retains authority over final local development decisions.

Contact Agency	Phone	Address
Local participating governments		Local participating governments

Website: See local participating governments

NCTCOG	817-640-3300	616 Six Flags Drive P.O. Box 5888 Arlington, TX 76005-5888

Website: http://www.dfwinfo.com/envir/trin/C_vision/safe/certificate.html

The Upper Trinity River CDC regulatory zone is comprised of the following six zones in the Metroplex:

- Upper Elm Fork
- West Fork Fort Worth
- West Fork East Fort Worth
- West Fork Mid Cities
- Confluence Area
- Lower Trinity Mainstem

Maps for each of the zones can be found at www.dfwinfo.com/envir/trin/cdc/maps/cdcmaps.html. The CDC regulatory zone and CDC review zone are represented in each map.

The **Corridor Development Certificate Manual** presents the standards for development within the Trinity River Corridor. The manual can be obtained from the North Central Texas Council of Governments or from any of the participating local governments. All local governments with development permitting authority within the river corridor are implementing the CDC process. The following governmental entities currently participate in the CDC process:

Cities

- Arlington
- Carrollton
- Coppell
- Dallas
- Farmers Branch
- Fort Worth
- Grand Prairie
- Irving
- Lewisville

Counties

- Dallas
- Denton
- Collin

Special Districts

- Tarrant Regional Water District
- Trinity River Authority

An exemption to the CDC process may be issued for specific activities outlined in the CDC Manual, including:

- Ordinary maintenance and repairs of any operational control structures,
- Outfall structures and associated intake structures where the outfall has been permitted under the Federal NPDES or State TPDES program,
- Discharge of material for backfill or bedding for utility lines, provided there is no significant change in pre-existing bottom contours and excess material is removed to an upland area, and
- Bank stabilization activities.

Under certain circumstances, the local government may issue a variance from the common permit criteria. Any development that cannot meet the established criteria detailed in the CDC Manual may request a variance. A variance shall be any modification of the literal provisions of the CDC Manual when strict enforcement of the CDC process would cause undue hardship, owing to circumstances unique to the individual property on which variance from the process is requested. Variances may also be issued for public projects deemed to be in the overall regional public interest, as determined by the jurisdiction's policy-making body.

An applicant seeking a variance must, like any other applicant, complete the CDC application form and undergo regional Review & Comment by signatories to the Interlocal Agreement. In addition, whenever a variance is requested, Preliminary Technical Review by the U.S. Army Corps of Engineers (USACE) is

required. The CDC process allows parallel review of the various federal, state, and local regulatory permits required for floodplain development to occur simultaneously. This feature of the CDC permit process ensures that no additional time is added to the local development decision-making process and that the overall federal, state, and local approval process is streamlined for quicker decision-making. Failure to comply with the provisions outlined in the CDC Manual will result in the penalties provided for under the floodplain management ordinance or regulations of the jurisdiction.

C.4.2 Hazard Mitigation Action Planning (HazMAP)

In response to a nationally identified need to reduce our vulnerability to disasters, the North Central Texas Council of Governments (NCTCOG) Department of Environment and Development, in cooperation with the Emergency Preparedness, Transportation, and Research & Information Services Departments, is coordinating multi-jurisdictional Hazard Mitigation Action Planning (HazMAP) for the 16-county region.

Hazard mitigation is the cornerstone of the Federal Emergency Management Agency's (FEMA) approach to reducing our nation's vulnerability to disasters. Hazard mitigation is defined as the actions taken to reduce or eliminate long-term risk to people and property from hazards and their effects. This definition distinguishes actions that have a long-term impact from those that are more closely associated with immediate preparedness, response, and recovery activities. Hazard mitigation is the only phase of emergency management specifically dedicated to breaking the cycle of damage, reconstruction, and repeated damage.

HazMAP is a multi-jurisdictional planning process consistent with the Disaster Mitigation Act of 2000 (DMA 2000) and the requirements of the State of Texas Division of Emergency Management (TxDEM) that is putting into place a framework for coordinated and focused hazard mitigation actions both at the local and regional levels. Important elements include:

- Guiding the effort is a HazMAP Review Team from among NCTCOG's many advisory committees, supported by an extensive outreach effort called HazMAP Partners, with government, private sector, and public representatives;
- Identifying and prioritizing the risks of a range of natural and technological hazards, from tornadoes and flooding to nuclear power plant accidents and chemical spills, as well as civil hazards;
- Considering where improved mitigation measures could help reduce hazard risks and vulnerability; and
- Working with our members to craft an initial five-year implementation strategy that will ensure continued local eligibility for federal funding.

The Stafford Act (amended by DMA 2000) requires locally-adopted and FEMA-approved mitigation action plans by November 1, 2004, in order for local governments to participate in certain federal grant programs for disaster planning and relief. Based on federal rules, the process includes annual reporting and plan updating at least every five years.

C.4.3 SAFE CLEAN & GREEN Regional Environmental Corridors

A strategic plan was adopted by NCTCOG in 2002 called SEE SAFE CLEAN & GREEN Regional Environmental Corridors, which lays a foundation for managing the region's waterways and greenways. The Vision of Success for the year 2025 is that all 2,400 miles of regional environmental corridors are SAFE CLEAN & GREEN.

The Vision of Success is further defined as:

- **SAFE** regional environmental corridors occur where the risks of loss of life and property damage from flooding are reduced, the natural function of the floodplain to safely convey storm water is preserved, and the financial integrity of the region's residents is secured.

- **CLEAN** regional environmental corridors support water quality that meets or exceeds the State's standards for public health and enjoyment, aquatic and wildlife protection, wastewater disposal needs, and sustains aesthetically attractive surface water conditions.
- **GREEN** regional environmental corridors are ribbons of greenways that tie together diverse ecological areas, open spaces, recreational and mobility opportunities throughout the region.

The strategic plan identifies opportunities for regional cooperation through a series of objectives accompanied by short-range strategic action recommendations to be undertaken in 2003 through 2007.

C.4.4 Regional Urban Storm Water Quality Management

Since 1989 the North Central Texas Council of Governments (NCTCOG) has been assisting municipalities and other entities to comply with storm water regulations issued by the Environmental Protection Agency (EPA) through a regional storm water management program. In 1990, National Pollutant Discharge Elimination System (NPDES) regulations were issued by EPA for point source discharges of storm water from municipal separate storm sewer systems (MS4s) of cities having populations of 100,000 or more. This was considered Phase I of the permitting program as outlined in Section C.3.1. In response, the seven largest cities in North Central Texas and two local districts of the Texas Department of Transportation formed a Regional Urban Storm Water Management Task Force to address many of the regulatory requirements collectively. NCTCOG served as coordinator of this regional program.

One of the application requirements was for municipalities to sample storm water from their MS4s to quantify pollutant loads and event mean concentrations. Upon approval from EPA Region 6, a network of 30 wet weather monitoring stations was set up in residential, commercial, highway, and industrial areas of Dallas, Fort Worth, Arlington, Garland, Irving, Plano, and Mesquite to sample seven storm events each for a total of 210 sample-events. These samples were analyzed for more than 190 parameters by the United States Geological Survey under contract with NCTCOG and the participating cities.

In the final analysis of the wet weather data, differences were found among the land use types. Residential outfalls had the highest levels of arsenic, diazinon, and chlordane - all of which are pesticides or major components of pesticides. Residential areas also had the highest levels of fecal bacteria indicating sewage related sources or pet waste. Not surprisingly, the nutrients phosphorus and nitrogen were also highest in storm water coming from residential areas. This is presumably indicative of excess fertilizer use. Nothing was characteristic of commercial areas probably since they tend to be a mixed assemblage of activities. From the industrial outfalls we saw an expected preponderance of metals, oil and grease, and other organics. Suspended solids were also prevalent since construction activity is considered industrial and most construction sites lacked adequate erosion control practices.

Subsequent analysis of these data yielded a list of "Constituents of Concern" (COC) based not on water quality standards (which are not directly applicable to storm water), but on frequency of detection in the storm water and on past history as a known water quality problem. The storm water COC list was split into Level 1 (highest concern) and Level 2 (lower priority). The high priority constituents of local concern in the Dallas-Fort Worth region are identified in Table C.4-1.

Table C.4-1 Storm Water Constituents of Local Concern

HIGH Priority	
Metals	**Others**
Cadmium	**Chlordane**
Chromium	**Diazinon**
Copper	**Fecal coliforms**
Lead	**Fecal streptococcus**
Zinc	**TSS**

In 1999, EPA issued draft regulations for Phase II of the NPDES permit program for MS4s. These

regulations impacted all remaining cities in urbanized areas along with certain designated others. The Texas Commission on Environmental Quality (TCEQ), who received delegation to administer the program in Texas, has issued a draft permit for storm water. Once the general permit is issued, regulated entities will be required to develop and implement comprehensive management programs to address urban storm water runoff over their initial five-year permit term.

Again in response to these regulations, NCTCOG expanded its storm water program in 1998 to include the smaller regulated entities and formed a Regional Storm Water Management Coordinating Council (RSWMCC) to oversee the effort. The RSWMCC first developed a Regional Strategy for Managing Storm Water Quality. This proactive policy position was intended to not only provide water quality benefits for the residents of the region, but to also comply with all applicable federal and state water quality regulations. Its stated goals for the expanded regional program are to:

- Protect the health and welfare of citizens and the environment,
- Effectively address state and federal regulations,
- Share professional knowledge and experience, and
- Provide training to governmental staff and the development community.

The Phase II regulations require regulated entities to specify what practices they will implement under six program elements. The strategy document focuses on these elements and encourages regional cooperation among participants in the program through the following series of cooperative initiatives:

- Public participation and education
- Control of construction site storm water runoff (iSWM Design Manual for Construction)
- Management of storm water impacts associated with post-construction (iSWM Design Manual for Site Development)
- Illicit discharge detection and elimination
- Municipal pollution prevention
- Regional cooperative monitoring

Through participation in the regional program and incorporation of these cooperative initiatives into their individual storm water management programs, participants can reduce their staff and administrative burdens while ensuring equity and consistency in storm water management practice across the region.

C.4.5 Center of Development Excellence

The DFW Region is expected to add around 3.5 million new residents by the year 2030, growing to over 9 million people. What effect will this growth have on transportation, open space, air quality, water quality, and water supply? How will this growth impact the quality of life for residents?

These are some of the questions being addressed by the Center of Development Excellence at the North Central Texas Council of Governments (NCTCOG). The Center of Development Excellence is a partnership of public and private interests formed to address the regional issues and infrastructure concerns of the future by fostering quality growth. Leaders in the fields of development, transportation, and government are working together to find cooperative and innovative development solutions to anticipated problems. Formed in 2002, the Center of Development Excellence is a comprehensive effort to maintain and enhance the DFW Metroplex's status as a desired location for people and businesses.

The mission of the Center of Development Excellence is to promote quality growth in North Central Texas that enhances the built environment, reduces vehicle miles of travel, uses water and energy resources effectively and efficiently, and helps advance environmental stewardship in order to ensure continued economic vitality and provide the highest attainable quality of life for all residents. Accordingly the Center

of Development Excellence recommends 10 Principles of Development Excellence as a guide to cities, counties, school districts, other public agencies, and the private sector as they plan and create future development and redevelopment in the region. The 10 Principles are:

1. **Development Options:** Provide a variety and balance of development options and land use types in communities throughout the region.
2. **Efficient Growth:** Foster redevelopment and infill of areas with existing infrastructure and promote the orderly and efficient provision of new infrastructure.
3. **Pedestrian Design:** Create more neighborhoods with pedestrian-oriented features, streetscapes, and public spaces.
4. **Housing Choice:** Sustain and facilitate a range of housing opportunities and choices for residents of multiple age groups and economic levels.
5. **Activity Centers:** Create mixed use and transit-oriented developments that serve as centers of neighborhood and community activity.
6. **Environmental Stewardship:** Protect sensitive environmental areas, preserve natural stream corridors, and create developments that minimize impact on natural features.
7. **Quality Places:** Strengthen community identity through use of compatible, quality architectural and landscape designs and preservation of significant historic structures.
8. **Transportation Efficiency:** Develop land uses, building sites, and transportation infrastructure that enhance the efficient movement of people, goods, and services.
9. **Resource Efficiency:** Provide functional, adaptable, and sustainable building and site designs that use water, energy, and material resources effectively and efficiently.
10. **Implementation:** Adopt comprehensive plans and ordinances that support Development Excellence and involve citizens and stakeholders in all aspects of the planning process.

The Center of Development Excellence is providing a forum for present-day government and business. Leaders must be just as visionary as were their predecessors if this region is to have the physical infrastructure and healthy environment needed for it to attain its full long-term potential. Regional cooperation will be crucial to maintaining that vitality. The dawning of the new century presents an opportune time to analyze our future requirements so that plans can be devised and implementation strategies can be developed which further guarantee that this region remains a leader among other regions of the nation.

APPENDIX

D

STORM WATER CONTROL DESIGN EXAMPLES

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D.5 Enhanced Swale Design Example D-55

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Introduction

The five storm water control examples included in Appendix D are presented to provide example computations using the most commonly used design standards. All of the possible options that might be incorporated into a storm water control structure are not covered in these examples.

For all possible design standards and options that might be incorporated into the design of a storm water control structure, refer to Chapter 1, Section 1.2 and especially to Table 1.2.1-1 on pages 1.2-1 & 1.2-2. In order to apply design standards to a given storm water control structure, additions or modifications to Chapter 1 must also be considered. These are found in the Local Criteria chapter of the Manual.

Local Criteria may determine the extent of downstream assessments; the return periods to be used for the "Streambank Protection" and "Conveyance" storms; and the acceptable design approach to be used for Water Quality Protection, Streambank Protection, and Flood Control. It is evident that there are many combinations of design options that might apply to a given storm water control structure.

Additional design examples will be developed for inclusion in Appendix D during the 2006 training efforts offered by NCTCOG.

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D.1 Storm Water Pond Design Example

The following design example is for a wet extended detention (ED) storm water pond. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). The layout of the Rolling Meadows Subdivision is shown in Figure 1.

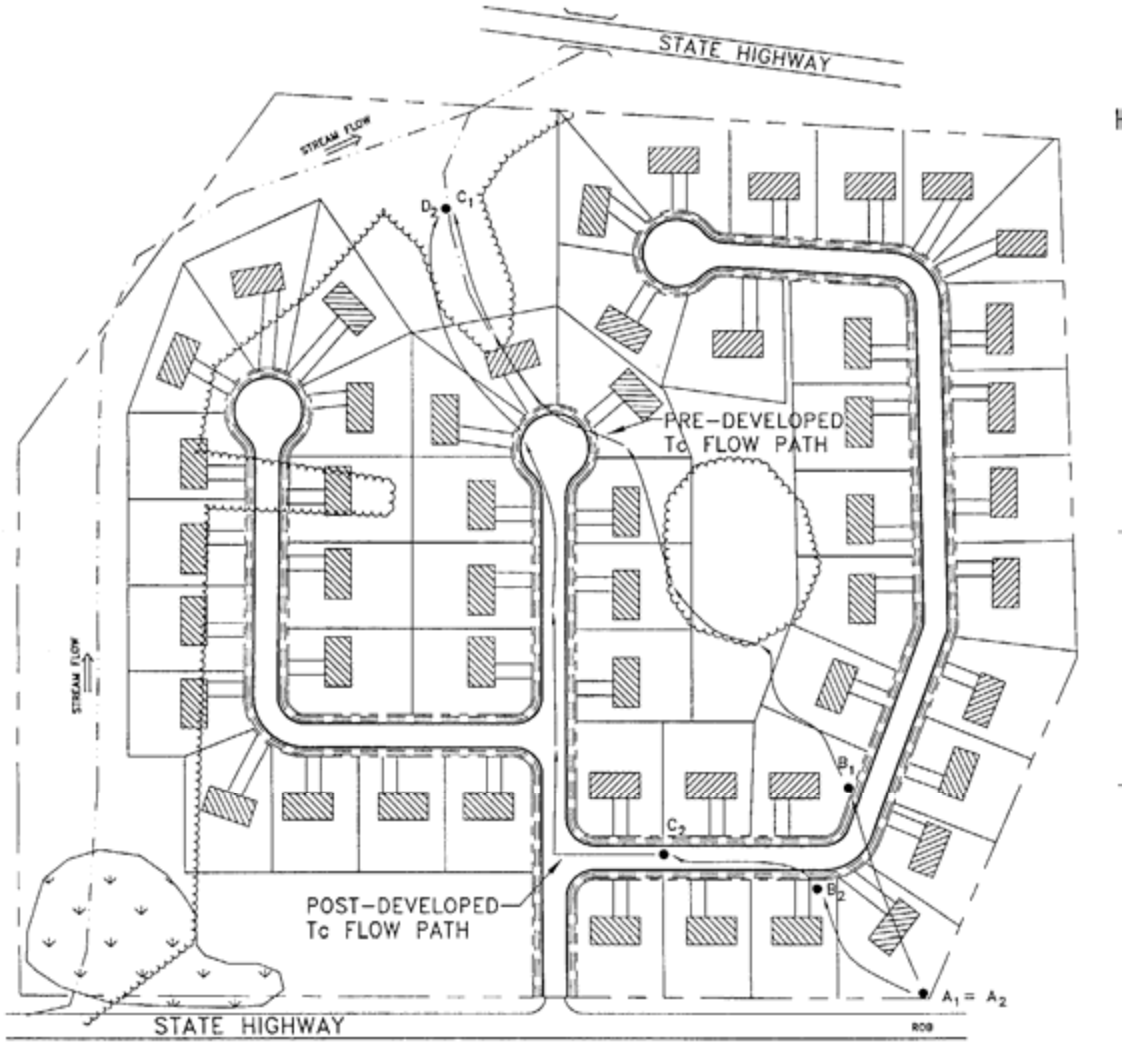

Figure 1. Rolling Meadows Site Plan

Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 38.0 ac		Pre	Post
Measured Impervious Area=13.8 ac; or I=13.8/38=36.3%	CN	76	85
Soils Types: 20% "C", 80% "B"	t_c	0.33 hr	0.19 hr
Zoning: Residential (½ acre lots) Denton County			

Computation of Preliminary Storm Water Storage Volumes and Peak Discharges

Step 1 -- Compute runoff control volumes from the *integrated* Design Approach

More details hydrologic calculations will be required during the design step – these numbers are preliminary.

Compute Water Quality Volume, WQ_v

- Compute Runoff Coefficient, R_v

  R_v = 0.05 + (I) (0.009)
  = 0.05 + (36.3) (0.009) = 0.38

- Compute WQ_v

  WQ_v = (1.5") (R_v) (A)
  = (1.5") (0.38) (38.0 ac) (1ft/12in)
  = 1.44 ac-ft

Develop Site Hydrologic and Hydrologic Input Parameters

Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	Area	CN	Tc
	Ac		*hrs*
pre-developed	38	76	0.33
post-developed	38	85	0.19

Perform Preliminary Hydrologic Calculations

Condition	$Q_{1\text{-}yr}$	$Q_{1\text{-}yr}$	$Q_{100\text{-}yr}$
Runoff	*Inches*	*cfs*	*cfs*
pre-developed	0.78	26.9	266
post-developed	1.29	61.3	402

Compute Streambank Protection Volume, (SP_v)
For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

Utilize SCS approach to Compute Streambank Protection Storage Volume

See Section 2.1

- Initial abstraction (I_a) for CN of 85 is 0.353: [I_a = (200/CN - 2)]
- I_a/P = (0.353)/ 2.64 inches = 0.13
- T_c = 0.19 hours
- q_u = 800 csm/in (Type II Storm)

Knowing q_u and T (extended detention time), find q_o/q_i. For a Type II rainfall distribution.

- Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.022
- $V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
- Where V_s equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.
- V_s/V_r = 0.65
- Therefore, $V_s = SP_v$ = 0.65(1.29")(1/12)(38 ac) = 2.66 ac-ft (116,077 cubic feet)

Define the average SP_v-ED Release Rate

- The above volume, 2.66 ac-ft, is to be released over 24 hours.
- (2.66 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 1.34 cfs

Analyze Safe Passage of 100 Year Design Storm (Q_f)

At final design, provide safe passage for the 100-year event, or detain it, depending on downstream conditions and local policy. Based on field observation and review of local requirements no control of the 100-year storm is necessary. If it were storage estimates would have been made similar to the Q_p Volume in the previous sub-step.

Table 1. Summary of General Storage Requirements for Rolling Meadows

Symbol	Control Volume	Volume Required (ac- ft)	Notes
WQ_v	Water Quality	1.44	
SP_v	Streambank Protection	2.66	Average extended detention release rate is 1.34 cfs over 24 hours
Q_f	Extreme Flood Protection	4.88	Detain to pre-developed conditions; Provide safe passage for the 100-year event in final design

The Modified Rational Method is used to estimate the storage volume.

Determine the Allowable Release Rate, Q_a

- Predevelopment Rational Coefficient, c_a= 0.45
- For t_c=0.33 hr, from Table A-3, i_{100} =6.99 in/hr
- From Equation 2.1.25, Q_a= c_a i A= (0.45) (6.99) (38)=119.5 cfs

Determine the Critical Duration of the Storm T_d

- From Table 2.1.7-1, a=325.18; b=24.822
- Post-developed Rational Coefficient, c=0.61
- From Equation 2.1.26, $Td=\sqrt{(2cAab/Q_a)} - b$

  $Td=\sqrt{(2(0.61)(38)(325.18)(24.822)/(119.5))} - (24.822) = 31.1$ min
- From Table A-3, for T_d= 31.1min, i_{Td} = 5.55 in/hr

  P_{Td} = 5.55 in/hr (31.1 min) (hr/ 60min) = 2.88 in

Determine the Allowable Release Rate, Q_a

- Predevelopment Rational Coefficient, c_a= 0.45
- For t_c=0.33 hr, from Table A-3, i_{100} =6.99 in/hr
- From Equation 2.1.25, Q_a= c_a i A= (0.45) (6.99) (38)=119.5 cfs

Compute Storage Volume

- Post-developed Time of concentration, t_c = 0.19 hr (60 min/hr) = 11.4 min
- From Equation 2.1.27a, V_{pre}= 60 $[cAa-(2cabAQ_a)^{1/2} + (Q_a/2)(b-t_c)]$

  V_{pre}= 60 $\{[0.61(38)(325.18)- 2(0.61)(24.822)(38)(119.5)]^{1/2} + (119.5/2)(24.822-11.4)\}$
  = 99154 ft^3
- From Equation 2.1.27a, V_{max} = V_{pre} * P_{180}/P_{Td}

P_{180} = 1.79 in/hr (3 hr) in Table A-3
V_{max} = 99154 (5.37/ 2.88) = 184881 ft^3 = 4.24 ac-ft

Experience has shown that additional 10-15% storage is required when multiple levels of extended detention are provided. So, for preliminary sizing purposes added 15% to the required volume for downstream flood control. O_f= 1.15 (4.24) = 4.88 ac-ft

PEAK DISCHARGE SUMMARY				
JOB:	P'TREE MEADOWS			EWB
DRAINAGE AREA NAME:	PRE-DEVELOPED CONDITIONS			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
meadow (good cond.)	D	0.5	78	30.40
meadow (good cond.)	C	0.5	71	2.60
woods (good cond.)	C	0.15	70	5.00
			AREA SUBTOTALS:	38.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.36"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	dense grass	'n'= 0.24	150ft	2.50%
				0.29 hrs
Shallow Flow	unpaved		500 ft	4.00%
			3.23 fps	0.04 hrs
Channel Flow				
Total Area in Acres =	38.00	Total Sheet Flow = 0.29 hrs.	Total Shallow Flow = 0.04 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	76			
Time of Concentration =	0.33 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.78	26.9
2 Year	3.36	1.26	45.1
5 Year	4.8	2.37	88
10 Year	5.52	2.97	113
25 Year	6.96	4.22	165
50 Year	7.92	5.09	205
100 Year	9.36	6.41	266

Figure 2. Rolling Meadows Pre-Development Conditions

PEAK DISCHARGE SUMMARY				
JOB:	P'TREE MEADOWS			EWB
DRAINAGE AREA NAME:	POST-DEVELOPED CONDITIONS			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
open space (good cond.)	D	0.5	80	20.00
woods (good cond.)	C	0.15	70	5.0
impervious area	D	.95	98	10.4
Impervious area	C	.95	98	2.6
			AREA SUBTOTALS:	38.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.36"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	short grass	'n'= 0.15	100 ft	2.50%
				0.15 hrs
Shallow Flow	paved		300 ft	2.00%
			2.87 fps	0.03 hrs
Channel Flow		'n'=0.013	600 ft	2.00%
Hydraulic Radius	X-S estimated	WP estimated	16.21 fps	0.01 hrs
Total Area in Acres =	38.00	Total Sheet Flow = 0.15 hrs.	Total Shallow Flow = 0.03 hrs.	Total Channel Flow = 0.01 hrs.
Weighted CN =	85			
Time of Concentration =	0.19 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.29	61.3
2 Year	3.36	1.89	92.3
5 Year	4.8	3.18	159
10 Year	5.52	3.85	196
25 Year	6.96	5.21	269
50 Year	7.92	6.14	322
100 Year	9.36	7.53	402

Figure 3. Rolling Meadows Post-Development Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of a storm water pond

Site Specific Data:

The site area and drainage area to the pond is 38.0 acres. Existing ground at the pond outlet is 919 MSL. Soil boring observations reveal that the seasonally high water table is at elevation 918. The underlying soils are predominantly clay and are suitable for earthen embankments and to support a wet pond without a liner. The stream invert at the adjacent stream is at elevation 916.

Other site screening aspects listed in Section 3.1 and 3.2.1 were assessed and a pond was found to be suitable.

Step 3 -- Confirm local design criteria and applicability

There are no additional requirements for this site.

Step 4 -- Determine pretreatment volume

Size wet forebay to treat 0.1"/impervious acre. (13.8 ac) (0.1") (1'/12") = **0.12 ac-ft**
(forebay volume is included in WQ_v as part of permanent pool volume)

Step 5 -- Determine permanent pool volume (and water quality extended detention volume)

Size permanent pool volume to contain 50% of WQ_v:
0.5 × (1.44 ac-ft) = **0.72 ac-ft**. (includes 0.12 ac-ft of forebay volume)

Size extended detention volume to contain 50% of WQ_v: 0.5 × (1.44 ac-ft) = **0.72 ac-ft**

Note: This design approach assumes that all of the extended detention volume will be in the pond at once. While this will not be the case, since there is a discharge during the early stages of storms, this conservative approach allows for extended detention control over a wider range of storms, not just the target rainfall.

Step 6 -- Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for permanent pool and water quality extended detention

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond. Storage must be provided for the permanent pool (including sediment forebays), extended detention (WQ_v-ED), SP_v-ED, downstream flood protection, plus sufficient additional storage to pass the 100-year storm with minimum freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. See Figure 4 for pond location on site, Figure 5 grading and Figure 6 for Elevation-Storage Data.

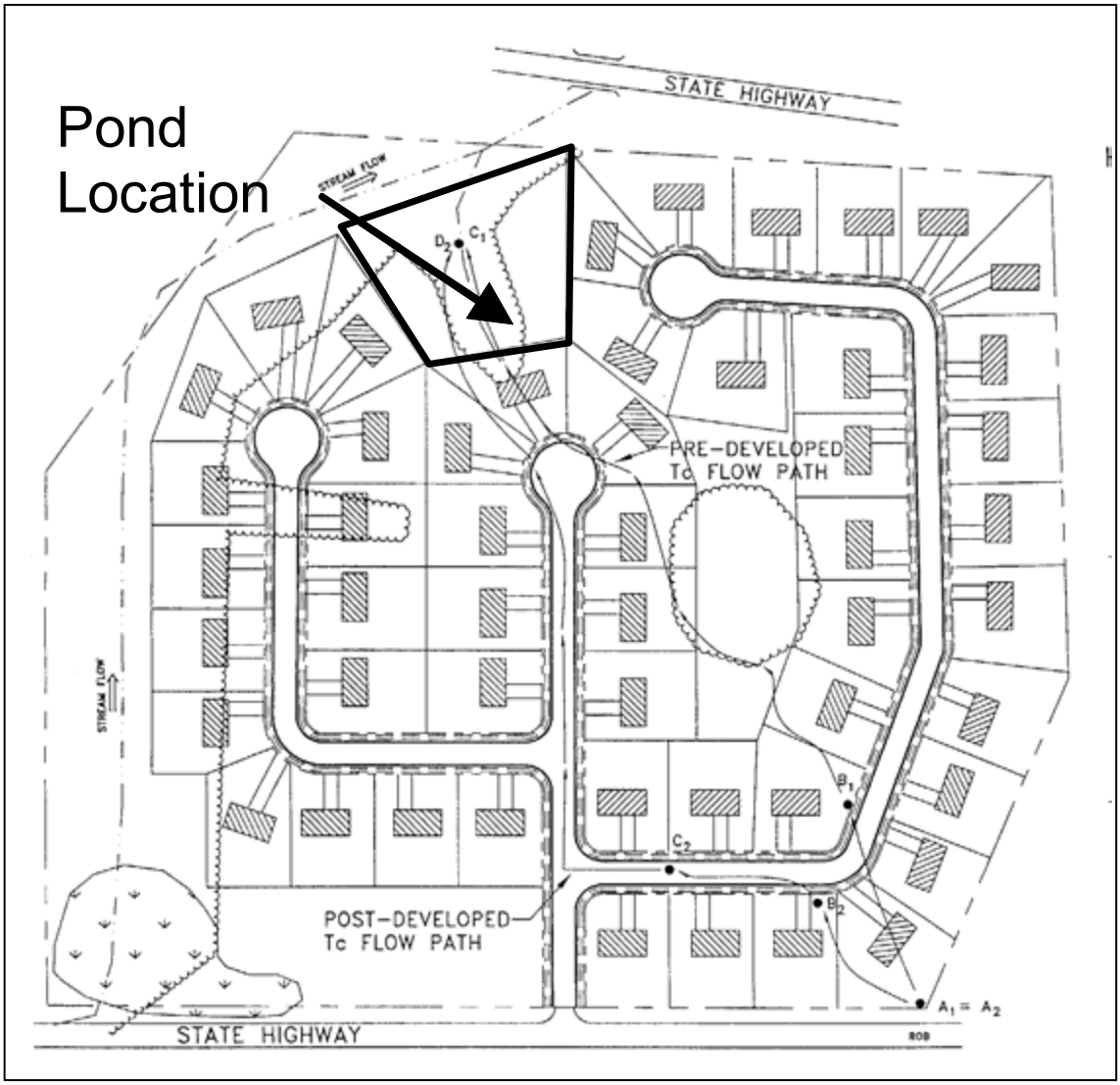

Figure 4. Pond Location on Site

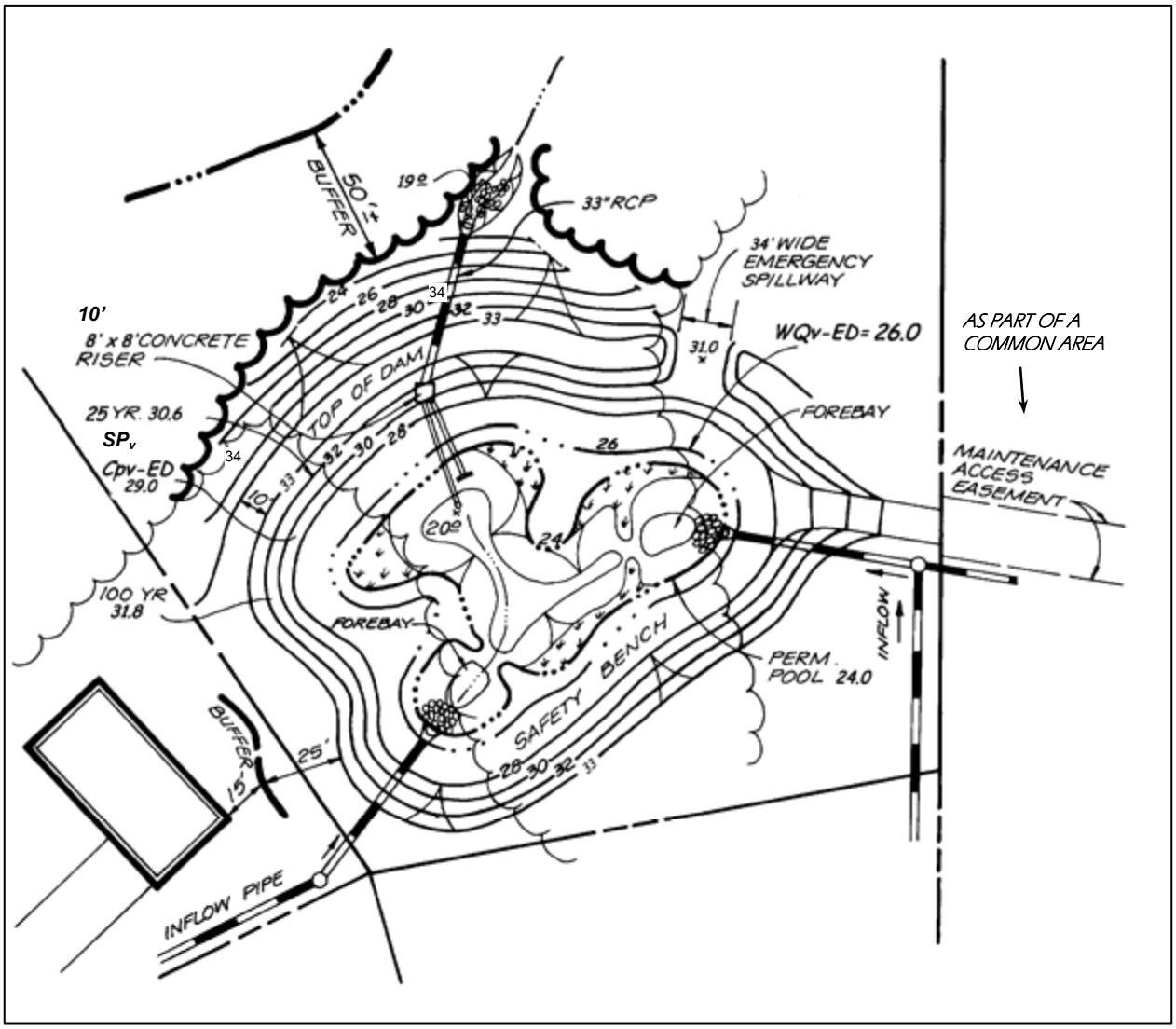

Figure 5. Plan View of Pond Grading (Not to Scale)

Elevation MSL	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume ac-ft	Volume Above Permanent Pool ac-ft
920.0						
921.0	7838	1	7838	7838	0.18	
923.0	11450	2	22900	30738	0.71	
924.0	14538	1	14538	45275	1.04	0
925.0	15075	1	15075	60350	1.39	0.35
925.5	16655	0.5	8328	68678	1.58	0.54
926.0	17118	0.5	8559	77236	1.77	0.73
926.5	21000	0.5	10500	87736	2.01	0.97
927.0	25000	0.5	12500	100236	2.30	1.26
927.5	30000	0.5	15000	115236	2.65	1.61
928.0	36000	0.5	18000	133236	3.06	2.02
928.5	38000	0.5	19000	152236	3.49	2.46
929.0	41000	0.5	20500	172736	3.97	2.93
929.5	43000	0.5	21500	194236	4.46	3.42
930.0	45000	0.5	22500	216736	4.98	3.94
930.5	47000	0.5	23500	240236	5.52	4.48
931.0	49000	0.5	24500	264736	6.08	5.04
931.5	52000	0.5	26000	290736	6.67	5.64
932.0	55000	0.5	27500	318236	7.31	6.27
932.5	58000	0.5	29000	347236	7.97	6.93
933.0	61000	0.5	30500	377736	8.67	7.63
933.5	65000	0.5	32500	410236	9.42	8.38
934.0	69000	0.5	34500	444736	10.21	9.17
935.0	74000	1	74000	518736	11.91	10.87

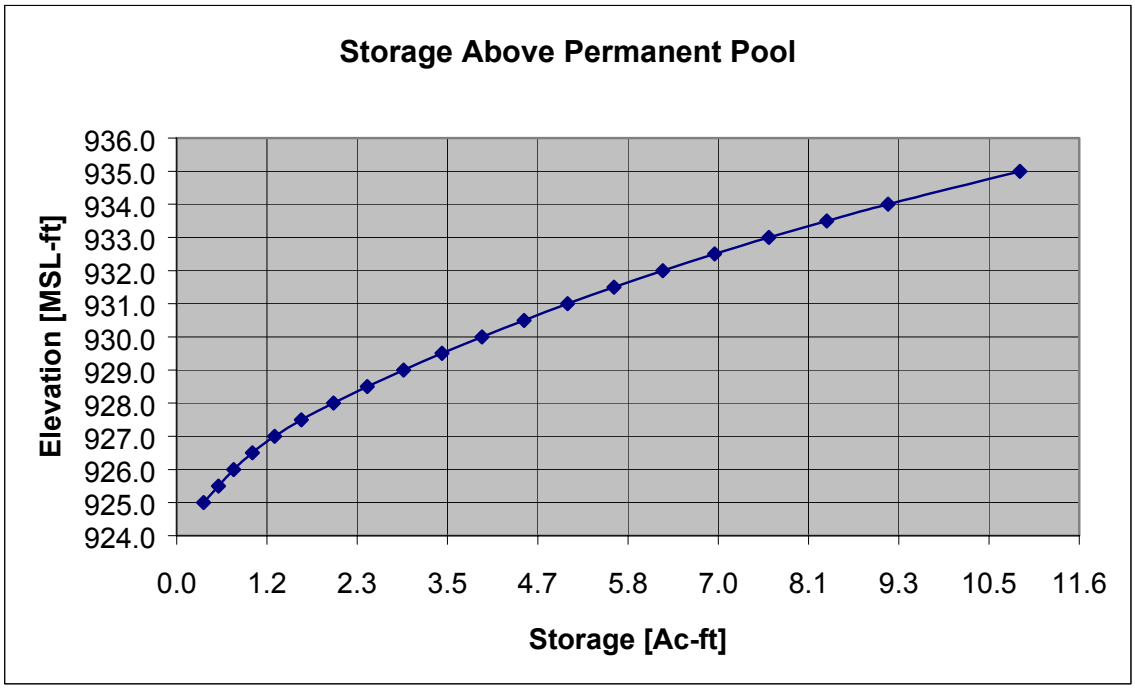

Figure 6. Storage-Elevation Table and Curve

Set basic elevations for pond structures

- The pond bottom is set at elevation 920.0.
- Provide gravity flow to allow for pond drain, set riser invert at 919.5.
- Set barrel outlet elevation at 919.0.

Set water surface and other elevations

- Required permanent pool volume = 50% of WQ_v = 0.72 ac-ft. From the elevation-storage table, read elevation 924.0 (1.04 ac-ft > 0.72 ac-ft). The site can accommodate it and it allows a small safety factor for fine sediment accumulation - OK
- Forebay volume provided in two pools with avg. vol. = 0.08 ac-ft each (0.16 ac-ft > 0.12 ac-ft) OK
- Required extended detention volume (WQ_v-ED) = 0.72 ac-ft. From the elevation-storage table (volume above permanent pool), read elevation 926.0 (0.73 ac-ft > 0.72 ac-ft) OK. Set extended detention wsel = 926.0

Note: Total storage at elevation 926.0 = 1.77 ac-ft (greater than required WQ_v of 1.44 ac-ft)

Compute the required WQ_v-ED orifice diameter to release 0.72 ac-ft over 24 hours

- Avg. extended detention release rate = (0.72 ac-ft)(43,560 ft^2/ac)/(24 hr)(3600 sec/hr) = 0.36 cfs
- Average head = (926.0 - 924.0)/ 2 = 1.0'
- Use orifice equation to compute cross-sectional area and diameter
  - $Q = CA(2gh)^{0.5}$, for Q = 0.36 cfs, h = 1.0 ft, C = 0.6 = discharge coefficient) solve for A
  - $A = 0.36 \text{ cfs} / [(0.6)((2)32.2 \text{ ft/s}^2)(1.0 \text{ ft}))^{0.5}]$ A = 0.075 ft^2, $A = \pi d^2 / 4$; dia. = 0.31 ft = 3.7"
  - Use 4" pipe with 4" gate valve to achieve equivalent diameter

Compute the stage-discharge equation for the 3.7" dia. WQv orifice

- $Q_{WQv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.075 \text{ ft}^2)\ [((2)(32.2 \text{ ft/s}^2))^{0.5}]\ (h^{0.5})$,
- $Q_{WQv\text{-}ED} = (0.36)\ h^{0.5}$, where: h = wsel - 924.16

(Note: account for one half of orifice diameter when calculating head)

Step 7 -- Compute extended detention orifice release rate(s) and size(s), and establish SP_v elevation

Set the SP_v pool elevation

- Required SP_v storage = 2.66 ac-ft (see Table 1).
- From the elevation-storage table, read elevation 929 (this includes the WQ_v).
- Set SP_v wsel = 929

Size SP_v orifice

- Size to release average of 1.34 cfs.
  - Average WQ_v-ED orifice release rate is 0.66 cfs, based on average head of 3.34' (926 – 924.16 + (929 – 926)/2)
  - SP_v-ED orifice release = 1.34 -0.66 = 0.66 cfs
- Head = (929 - 926.0)/2 = 1.5'

Use orifice equation to compute cross-sectional area and diameter

- $Q = CA(2gh)^{0.5}$, for h = 1.5'
  - A = 0.68 cfs / $[(0.6)((2)(32.2'/s^2)(1.5'))^{0.5}]$
  - A = 0.12 ft^2, $A = \pi d^2 / 4$;
  - dia. = 0.38 ft = 4.6"
  - Use PVC pipe to the nearest 1" (in this case 5" PVC pipe)

Compute the stage-discharge equation for the 4.6" dia. SP_v orifice

- $Q_{SPv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.12\ ft^2)\ [((2)\ (32.2'/s^2))^{0.5}]\ (h^{0.5})$,
- $Q_{SPv\text{-}ED} = (0.55)\ (h^{0.5})$, where: h = wsel - 926.19

(Note: Use the distance form the water surface to the center of the orifice when calculating head)

Step 8 -- Calculate Q_f (100-year storm) release rate and water surface elevation

In order to calculate the release rate and water surface elevation, the designer must set up a stage-storage-discharge relationship for the control structure for each of the low flow release pipes (WQ_v-ED and SP_v-ED) plus the 100 year storm.

Develop basic data and information

- The 100 year pre-developed peak discharge = 266 cfs,
- The post developed inflow = 402 cfs, from Table 1,
- From previous estimate Q_f = 4.88 ac-ft.
- From elevation-storage table (Figure 6), read elevation 930.9.

Size 100-year slot to release 266 cfs at elevation 930.9.

- @ wsel 930.9:
  - WQ_v-ED orifice releases 0.93 cfs,
  - SP_v-ED orifice releases 1.19 cfs, therefore;
  - Allowable Q_p = 266 cfs - (.93 + 1.19) = 263.9 cfs, say 264 cfs.
- Max head = (930.9 – 929) = 1.9'
- Use weir equation to compute slot length
  - $Q = CLH^{3/2}$
  - L = 264 cfs / (3.1) $(1.9^{3/2})$ = 32.5 ft
- Use four 8.5 ft x 2 ft slots for 100-year release (opening should be slightly larger than needed so as to have the barrel control before slot goes from weir flow to orifice flow).

Check orifice equation using cross-sectional area of opening

- $Q = CA(2gh)^{0.5}$, for h = 1.0' (For orifice equation, h is from midpoint of slot)
- A = 4 (8.5') (2') = 68.0 ft^2
- Q = 0.6 $(68.0ft^2)\ [(64.4)(1.0)]^{0.5}$ = 327 cfs > 266 cfs, so use weir equation

$Q_{100} = (3.1)\ (34')\ H^{3/2}$, $Q_{100} = (105.8)\ H^{3/2}$, where H = wsel – 929.0

Size barrel to release approximately 266 cfs at elevation 930.9

- Check inlet condition: (use Section 4.2 culvert charts)
  - H_w = 930.9-919.5 = 11.4 ft
  - Try 60" diameter RCP, Using Figure 4.2-2(a) with entrance condition 1
  - H_w / D = 11.4/5 = 2.28, Discharge = 280 cfs
- Check outlet condition:
  - $Q = a\,[(2gH)/(1+k_m+k_pL)]^{0.5}$
    where:
    Q = discharge in cfs
    a = pipe cross sectional area in ft^2
    g = acceleration of gravity in ft/sec^2
    H = head differential (wsel - downstream centerline of pipe or tailwater elev.)
    k_m = coefficient of minor losses (use 1.0)
    k_p = pipe friction loss coef. (= $5087n^2/d^{4/3}$, d in ", n is Manning's n)
    L = pipe length in ft
  - H = 930.9 - (919.0 + 2.5) = 9.4'
  - for 60" RCP, 70 feet long:
  - $Q = 19.63\,[(64.4)\,(9.4) / 1+1+(.003)\,(70))]^{0.5}$ = 324.9 cfs
  - 280 cfs < 325 cfs, so barrel is inlet controlled.

Note: Pipe will control flow before high stage inlet reaches max head.

Complete stage-storage-discharge summary (Figure 7) up to preliminary 100-year wsel (930.9) and route 100 year post-developed condition inflow using computer software.

Elevation MSL	Storage ac-ft	Flow Flow WQv-ED 3.7" eq dia		Riser						Barrel				Emergency Spillway		Total Discharge
				SPv-ED 4.7" eq dia		High Stage Slot				Inlet		Pipe				
						Orifice		Weir								
		H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	Q cfs
924.0	0.00	0	0													0
925.0	0.35	0.8	0.33													0.33
925.5	0.54	1.3	0.42													0.42
926.0	0.73	1.8	0.49	0	0											0.49
926.5	0.97	2.3	0.55	0.3	0.31											0.86
927.0	1.26	2.8	0.61	0.8	0.50											1.11
927.5	1.61	3.3	0.66	1.3	0.63											1.29
928.0	2.02	3.8	0.71	1.8	0.74											1.45
928.5	2.46	4.3	0.75	2.3	0.84											1.59
929.0	2.93	4.8	0.79	2.8	0.92	N/A		0.0	0.0							1.71
929.5	3.42	5.3	0.83	3.3	1.00			0.5	37.4							39.2
930.0	3.94	5.8	0.87	3.8	1.07			1.0	105.8							107.7
930.5	4.48	6.3	0.91	4.3	1.14			1.5	194.4							196.5
930.9	4.93	6.7	0.93	4.7	1.19			1.9	277.1	11.4	280.0	9.4	324.9			279.2
931.0	5.04	-	-	-	-	1.0	327.0	2.0	299.2	11.5	280.0	9.5	326.6	0.0	0.0	280.0
931.5	5.64	-	-	-	-					12.0	285.0	10.0	335.1	0.5	24.0	309.0
932.0	6.27	-	-	-	-					13.0	290.0	10.5	343.4	1.0	79.0	369.0
932.5	6.93	-	-	-	-					13.5	300.0	11.0	351.4	1.5	154.0	454.0

Figure 7 Stage-Storage-Discharge Summary

Note: Adequate outfall protection must be provided in the form of a riprap channel, plunge pool, or combination to ensure non-erosive velocities. Plans must indicate pipe class, joint type, and bedding.

Step 9 -- Design embankment(s) and spillway(s)

Set the emergency spillway at elevation 931.0 and use design information and criteria Earth Spillways (not included in this manual)

- Q_{100} inflow = 402 cfs.
- Try 34' wide vegetated emergency spillway with 3:1 side slopes.
  - @ elevation 932.6, H = 1.5', Emergency spillway, Q_{ES} = 154 cfs. Primary spillway, Q_{PS} . 300 cfs
  - Q_{ES} + Q_{PS} = 454 cfs, will be able to safely convey Q_f = 402. (use computer routing for exact elevations and discharges).
  - 100 year wsel = 932.2, say 932.5, so set top of embankment with 1 foot of freeboard at elevation 933.5.

Step 10 -- Investigate potential pond hazard classification

Refer to Texas Commission on Environmental Quality (TCEQ) Dam Safety Program to establish preliminary classification of embankment and whether special design criteria need to be met. Their regulations apply for the construction of dams that are six feet or more in height.

Check pond classification: Height = 931 -919 = 12', equals assumed embankment height, Pond will remain Category II or lower.

As reported in Table 1, the preliminary maximum storage volume required is about 4.24 acre-feet. Therefore, for initial design considerations, no additional dam safety requirements will apply. Once final design elevations and storage volumes have been determined, a final check for dam rules exemption should be made by the designer.

Step 11. -- Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

Table 2 Summary of Controls Provided

Control Element	Type/Size of Control	Storage Provided	Elevation	Discharge	Remarks
Units		*Acre-feet*	*MSL*	*cfs*	
Permanent Pool		0.86	924.0	0	part of WQ_v
Forebay	submerged berm	0.12	924.0	0	included in permanent pool volume
Water Quality Extended Detention (WQ_v-ED)	4" pipe, sized to 3.7" equivalent diameter	0.72	926.0	0.36	part of WQ_v above perm. pool, discharge is average release rate over 24 hours
Streambank Protection (SP_v-ED)	6" pipe sized to 5.0" equivalent diameter	2.66	929.0	1.34	volume above perm. pool, discharge is average release rate over 24 hours
Downstream Flood Protection (Q_p)	Four 8.5' x 2' slots on a 10' x 8' riser, 60" barrel.	4.88	931.0	280	volume above perm. pool
Extreme Flood Protection ($Q_{f\text{-}100}$)	34' wide earth spillway	6.53	932.2	207	volume above perm. pool

See Figure 8 for profile through principal spillway of the facility.
See Figure 9 for a schematic of the riser.

Step 12 -- Prepare Vegetation and Landscaping Plan

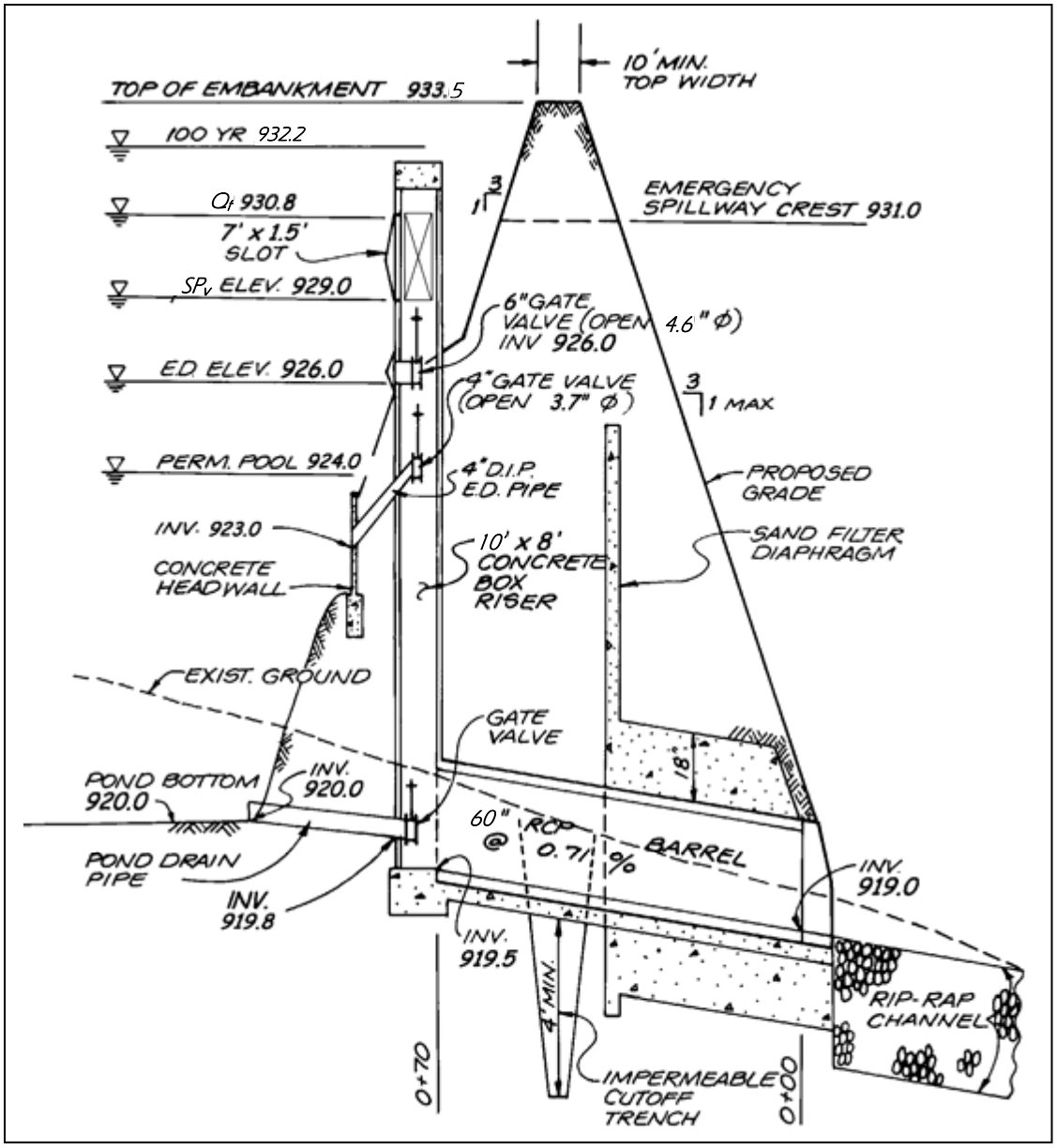

Figure 8. Profile of Principal Spillway

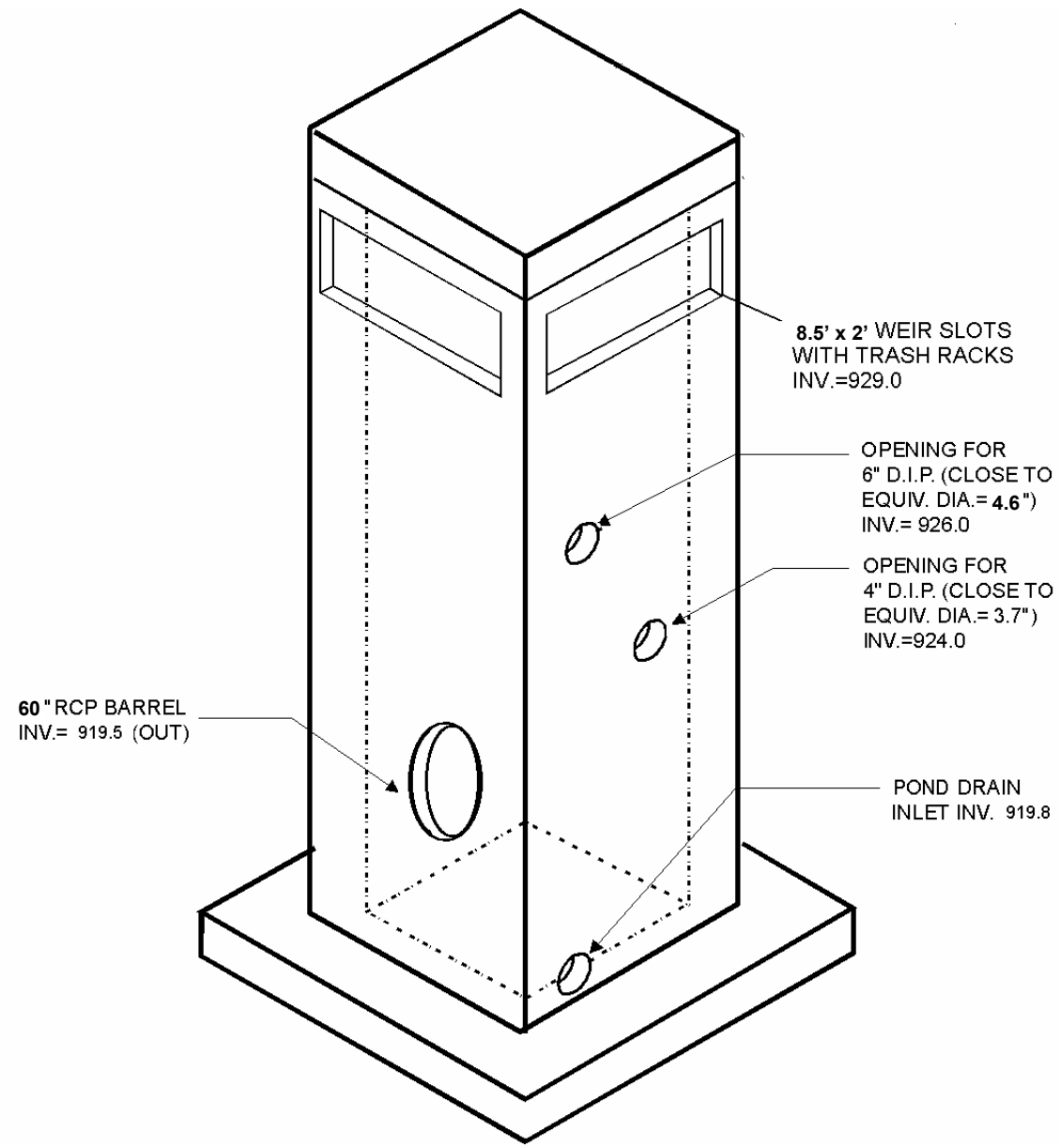

Figure 9. Schematic of Riser Detail

D.2 Bioretention Area Design Example

This example focuses on the design of a bioretention facility to meet the water quality treatment requirements of the Wellington Recreation Center. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of bioretention is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility or pass through the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). Under some conditions, Streambank Protection storage can be provided by bioretention facilities. The layout of the Wellington Recreation Center is shown in Figure 1.

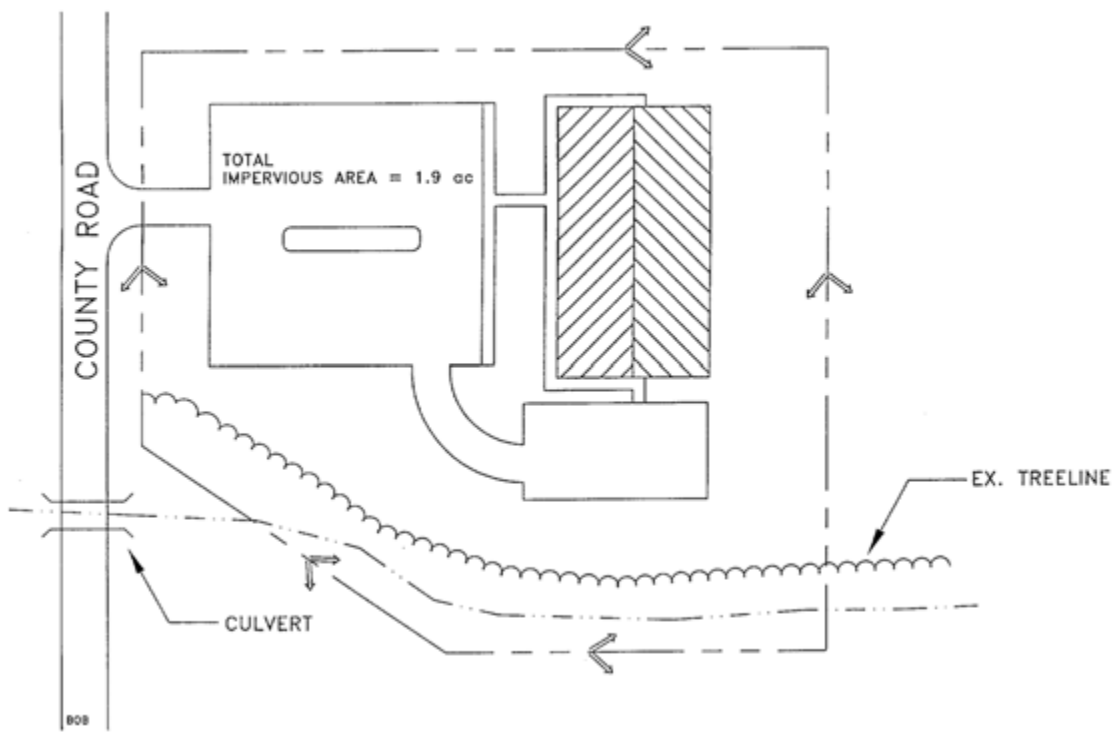

Figure 1. Wellington Recreation Center Site Plan

Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac		Pre	Post
Measured Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	77	91
Soils Type "D" Collin County	t_c	0.41 hr	0.20 hr

Computation of Preliminary Storm Water Storage Volumes and Peak Discharges

Step 1 -- Compute runoff control volumes from the *integrated* Design Approach

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.5") (R_v) (A) / 12$
  $= (1.5") (0.62) (3.0ac) (43{,}560ft^2/ac) (1ft/12in)$
  $= 8{,}102\ ft^3$

Compute Stream Streambank Protection Volume (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for streambank protection and flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 2.1 to determine pre- and post-development peak discharges for the 1-yr, and 100-yr 24-hour return frequency storms.

- Per attached TR-55 calculations (Figures 2 and 3)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*
Pre-developed	77	0.83	2.1	21
Post-Developed	91	1.79	6.7	37

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (I_a) for CN of 91 is 0.27: [$I_a = (200/CN - 2)$]

  $I_a/P = (0.198)/ 2.64$ inches $= 0.075$
  $T_c = 0.20$ hours
  $q_u = 820$ csm/in

Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.022

For a Type II rainfall distribution,

$V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

$V_s/V_r = 0.65$

Therefore, $V_s = SP_v = 0.65(1.74")(1/12)(3\ ac) = 0.28\ ac\text{-}ft = 12{,}317\ ft^3$

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the 25 year storm flood protection control.

Table 1. Summary of General Design Information for Wellington Recreation Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	12,317	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	Wellington Recreation Center			EWB
DRAINAGE AREA NAME:	Pre-Developed Conditions			
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
woods (good cond.)	D		77	3.0
		AREA SUBTOTALS:		3.0
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.6"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	dense grass	'n'= 0.24	150 ft	1.50%
				0.35 hrs
Shallow Flow	unpaved		500 ft	2.00%
			2.28 fps	0.06 hrs
Channel Flow				
Total Area in Acres =	3.00	Total Sheet Flow = 0.35 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	77			
Time of Concentration =	0.41 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.83	2.1
2 Year	3.60	1.51	4.0
5 Year	5.04	2.66	7.2
10 Year	6.00	3.48	9.6
25 Year	7.20	4.55	13.0
50 Year	8.40	5.64	16.0
100 Year	9.84	6.99	21.0

Figure 2. Wellington Recreation Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Wellington Recreation Center			EWB
DRAINAGE AREA NAME:	Post-Development Conditions			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
open space (good cond.)	D		80	0.50
woods (good cond.)	D		70	0.60
impervious area	D		98	1.90
		AREA SUBTOTALS:		38.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.6"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	dense grass	'n'= 0.24	50 ft	1.50%
				0.14 hrs
Shallow Flow	paved		600 ft	2.00%
			2.87 fps	0.06 hrs
Channel Flow		'n'=0.024	50 ft	2.00%
Hydraulic Radius	X-S estimated	WP estimated	7.25 fps	0.00 hrs
Total Area in Acres =	3.00	Total Sheet Flow = 0.14 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	91			
Time of Concentration =	0.20 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.74	6.7
2 Year	3.60	2.64	10.4
5 Year	5.04	4.02	16.0
10 Year	6.00	4.96	20.0
25 Year	7.20	6.14	25.0
50 Year	8.40	7.32	31.0
100 Year	9.84	8.75	37.0

Figure 3. Wellington Recreation Center Post-Developed Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of a bioretention area.

Site Specific Data:

Existing ground elevation at the facility location is 922.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 913.0 feet and underlying soil is predominately clay. Adjacent creek invert is at 912.0 feet.

Step 3 -- Confirm local design criteria and applicability

There are no additional local criteria that must be met for this design.

Step 4 -- Compute WQ_v peak discharge (Q_{wq})
Step 5 -- Size flow diversion structure, if needed

Bioretention areas can be either on or off-line. On-line facilities are generally sized to receive, but not necessarily treat, the 25-year event. Off-line facilities are designed to receive a more or less exact flow rate through a weir, channel, manhole, "flow splitter", etc. This facility is situated to receive direct runoff from grass areas and parking lot curb openings and piping for the 25-year event (25.0 cfs), and *no special flow diversion structure is incorporated.*

Step 6 -- Determine size of bioretention ponding / filter area

$A_f = (WQ_v)(d_f) / [(k)(h_f + d_f)(t_f)]$

where:
A_f = surface area of filter bed (ft^2)
d_f = filter bed depth (ft)
k = coefficient of permeability of filter media (ft/day)
h_f = average height of water above filter bed (ft)
t_f = design filter bed drain time (days) (48 hours is recommended)

$A_f = (8{,}102\ ft^3)(5') / [(0.5'/day)(0.25' + 5')(2\ days)]$ (With $k = 0.5'/day$, $h_f = 0.25'$, $t_f = 2$ days)

A_f = 7,716 sq ft

Step 7 -- Set design elevations and dimensions of facility

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 7,716 sq ft, say facility is roughly 65' by 120'. See Figure 5. Set top of facility at 921.0 feet, with the berm at 922.0 feet. The facility is 5' deep, which will allow 3' of freeboard over the seasonally high water table. See Figure 6 for a typical section of the facility.

Step 8 -- Design conveyance to facility (off-line systems)

This facility is not designed as an off-line system.

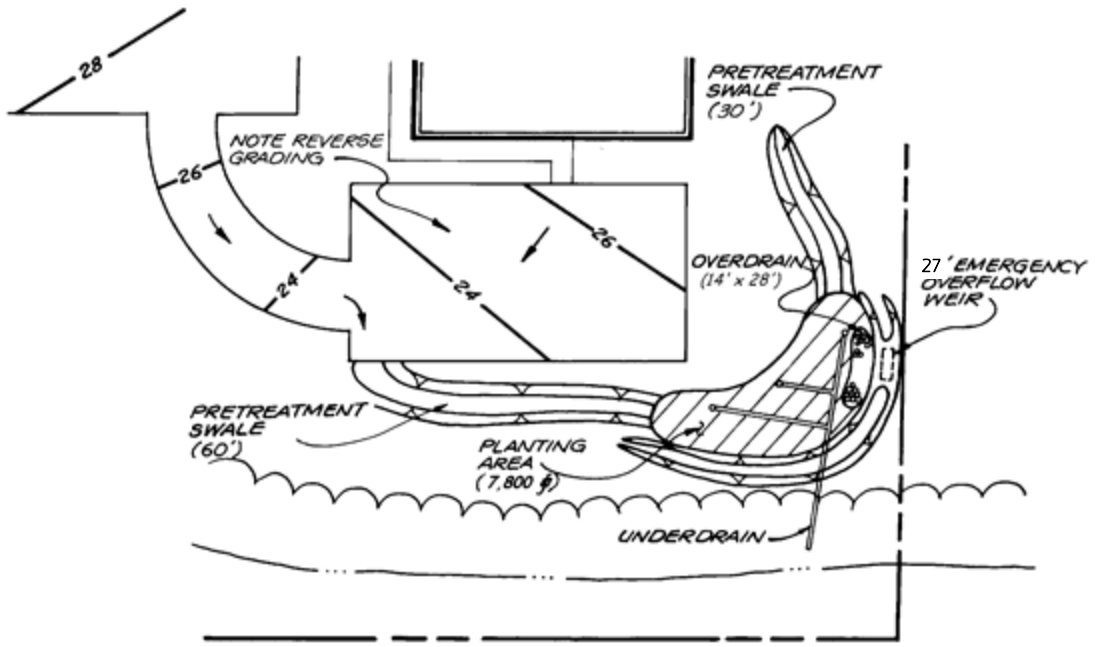

Figure 5. Plan View of Bioretention Facility

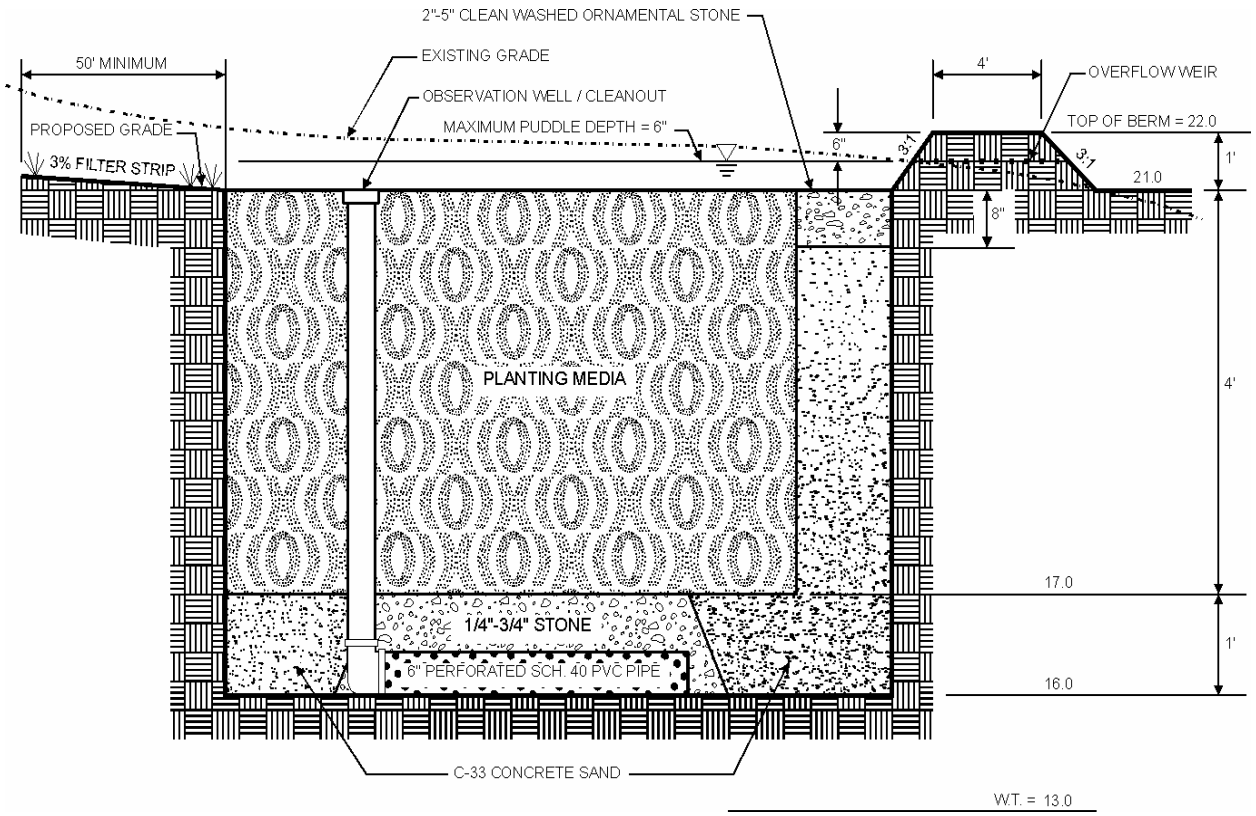

Figure 6. Typical Section of Bioretention Facility

Step 9 -- Design pretreatment

Pretreat with a grass channel, based on guidance provided in Table 5.2.3-1, below. For a 3.0 acre drainage area, 63% imperviousness, and slope less than 2.0%, provide a 90' grass channel at 1.5% slope. The value from Table 5.2.3-1 is 30' for a one acre drainage area.

Table 5.2.3-1 Pretreatment Grass Channel Guidance for 1.0 Acre Drainage Area
(Adapted from Claytor and Schueler, 1996)

Parameter	≤ 33% Impervious		Between 34% & 66% Impervious		≥ 67% Impervious		Notes
Slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Max slope = 4%
Grassed channel min. length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

Step 10 -- Size underdrain area

Base underdrain design on 10% of the A_f or 772 sq ft. Using 6" perforated plastic pipes surrounded by a three-foot-wide gravel bed, 10' on center (o.c.). See Figures 5 and 6.

(772 sq ft)/3' per foot of underdrain = 257', say 260' <u>of perforated underdrain</u>

Step 11 – Design emergency overflow

To ensure against the planting media clogging, design a small ornamental stone window of 2" to 5" stone connected directly to the sand filter layer. This area is based on 5% of the A_f or 386 sq ft. Say 14' by 28'. See Figures 5 and 6.

The parking area, curb and gutter are sized to convey the 25-year event to the facility. Should filtering rates become reduced due to facility age or poor maintenance, an overflow weir is provided to pass the 25-year event. Size this weir with 6" of head, using the weir equation.

$Q = CLH^{3/2}$

Where C = 2.65 (smooth crested grass weir)
Q = 25.0 cfs
H = 6"

Solve for L: $L = Q / [(C)(H^{3/2})]$ or $(25.0 \text{ cfs}) / [(2.65)(.5)^{1.5}] = 26.7'$ <u>(say 27')</u>

Outlet protection in the form of riprap or a plunge pool/stilling basin should be provided to ensure non-erosive velocities. See Figures 5 and 6.

Step 12 – Prepare Vegetation and Landscaping Plan

Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Select species locations (i.e., on center planting distances) so species will not "shade out" one another. Do not plant trees and shrubs with extensive root systems near pipe work. A potential plant list is presented in Appendix F.

D.3 Sand Filter Design Example

This example focuses on the design of a surface sand filter to meet the water quality treatment requirements of the Falcon Creek Community Center. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Falcon Creek Community Center is shown in Figure 1.

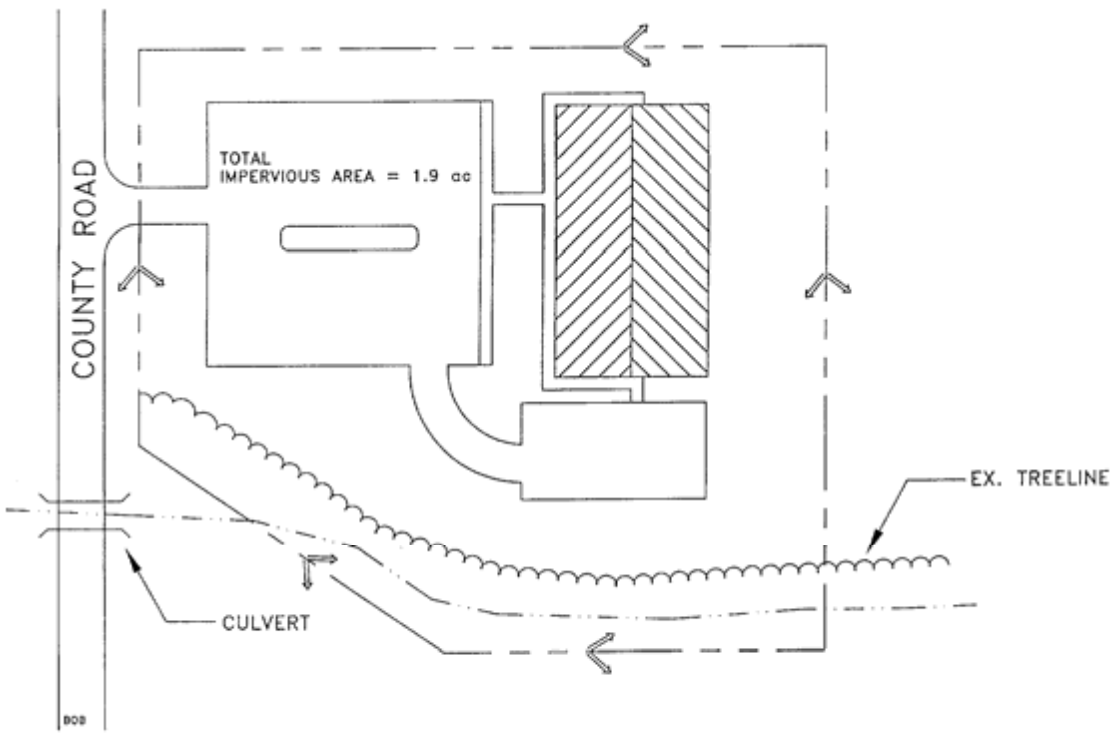

Figure 1. Falcon Creek Community Center Site Plan

Base Data		Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac			Pre	Post
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%		CN	78	91
Soils Type "D"	Dallas County	t_c	0.41hr	0.16 hr

Computation of Preliminary Storm Water Storage Volumes and Peak Discharges

Step 1 -- Compute runoff control volumes from the *integrated* Design Approach

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.5")(R_v)(A) / 12$
  $= (1.5")(0.62)(3.0\ ac)(43{,}560\ ft^2/ac)(1ft/12in)$
  $= 8{,}102\ ft^3 = 0.186$ ac-ft

Compute Stream Streambank Protection Volume, (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

- Develop Site Hydrologic and Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations

  Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*
Pre-developed	78	0.88	2.2	20.0
Post-Developed	91	1.74	7.3	39.0

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (I_a) for CN of 91 is 0.198: (TR-55) [$I_a = (200/CN - 2)$]

  Ia/P = (0.198)/ 2.64 inches = 0.075
  T_c = 0.16 hours
  q_u = 900 csm/in

Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.02

$V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

Vs/Vr = 0.655

Therefore, $V_s = SP_v = 0.655(1.74")(1/12)(3\ ac) = 0.285$ ac-ft = 12,415 ft^3

- Define the average extended detention Release Rate

  The above volume, 0.30 ac-ft, is to be released over 24 hours.
  (0.30 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 0.15 cfs

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the 25-year storm flood protection control.

Table 1. Summary of General Design Information for Falcon Creek Community Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	12,415	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	Falcon Creek Center			EWB
DRAINAGE AREA NAME:	Pre-Developed Conditions			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
meadows (good cond.)	D		78	2.40
woods (good cond.)	D		77	0.60
		AREA SUBTOTALS:		3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.6"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	Dense grass	'n'= 0.24	150 ft	2.50%
				0.35 hrs
Shallow Flow	unpaved		500 ft	2.00%
			2.28 fps	0.06 hrs
Channel Flow				
Hydraulic Radius				
Total Area in Acres =	3.00	Total Sheet Flow = 0.35 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	78			
Time of Concentration =	0.41 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.88	2.2
2 Year	3.60	1.57	4.2
5 Year	4.8	2.54	6.9
10 Year	5.76	3.37	9.3
25 Year	7.20	4.66	13.0
50 Year	8.40	5.76	17.0
100 Year	9.6	6.98	20.0

Figure 2. Falcon Creek Community Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Falcon Creek Center			EWB
DRAINAGE AREA NAME:	Post-Developed Conditions			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
open space (good cond.)	D		78	0.50
woods (good cond.)	D		77	0.60
impervious	D		98	1.90
			AREA SUBTOTALS:	3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.6"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	short grass	'n'= 0.15	50 ft	1.50%
				0.10 hrs
Shallow Flow	paved		600 ft	2.00%
			2.87 fps	0.06 hrs
Channel Flow		'n'= 0.024	50 ft	2.00%
Hydraulic Radius= 0.75	X-S estimated	WP estimated	7.25 fps	0.00hrs
Total Area in Acres =	3.00	Total Sheet Flow = 0.10 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	91			
Time of Concentration =	0.16 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.74	7.3
2 Year	3.60	2.64	11.0
5 Year	4.8	3.79	17.0
10 Year	5.76	4.72	21.0
25 Year	7.20	6.14	27.0
50 Year	8.40	7.32	33.0
100 Year	9.6	8.51	39.0

Figure 3. Falcon Creek Community Center Post-Developed Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of a surface sand filter.

Site Specific Data:

Existing ground elevation at the facililty location is 22.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 13.0 feet. Adjacent creek invert is at 12.0.

Step 3 -- Confirm local design criteria and applicability.

There are no additional requirements for this site.

Step 4 -- Compute WQ_v peak discharge (Q_{wq}) & Head

- Water Quality Volume:

  WQ_v previously determined to be 8,102 cubic feet.

- Determine available head (See Figure 5)

  Low point at parking lot is 23.5. Subtract 2' to pass Q_{100} discharge (39) and a half foot for channel to facility (21.0). Low point at stream invert is 12.0. Set outfall underdrain pipe 2' above stream invert and add 0.5' to this value for drain (14.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (16.67). The total available head is 21.0 - 16.67 or 4.33 feet. Therefore, the average depth, h_f, is (h_f) = 4.33' / 2, and h_f = 2.17'.

  The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. Conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

  The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

- Using the water quality volume (WQ_v), a corresponding Curve Number (CN) is computed utilizing the following equation:

  $$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\,QP)^{1/2}]$$

  where P = rainfall, in inches (use 1.5" for the Water Quality Storm)
  and Q = runoff volume, in inches (equal to $WQ_V \div$ area)

- Once a CN is computed, the time of concentration (t_c) is computed
- Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the Water Quality Storm is computed (based on the procedures identified in Section 2.1 (typically Type II in the North Central Texas region).
  - Read initial abstraction (I_a), compute I_a/P
  - Read the unit peak discharge (q_u) for appropriate t_c

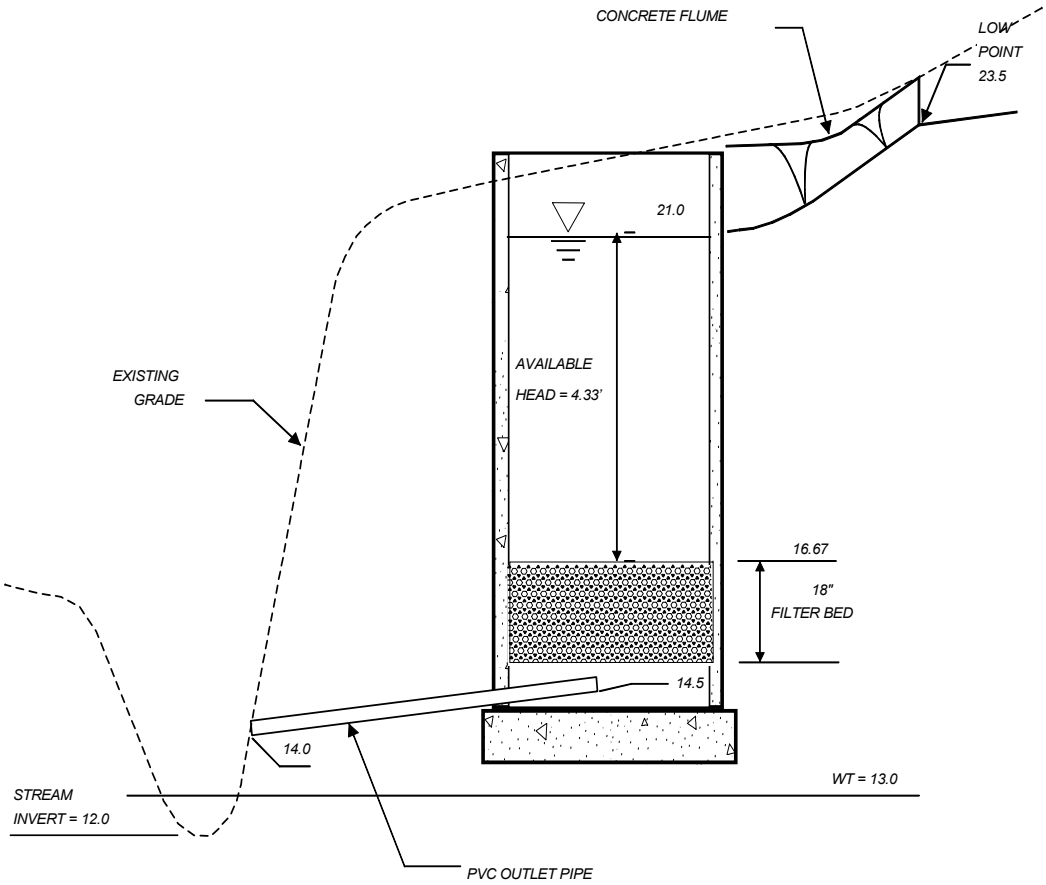

Figure 5. Available Head Diagram

- Using the water quality volume (WQ_v), compute the water quality peak discharge (Q_{wq})

  $\mathbf{Q_{wq} = q_u{*}A{*}WQ_V}$

  where

  Q_{wq} = the peak discharge, in cfs

  q_u = the unit peak discharge, in cfs/mi²/inch

  A = drainage area, in square miles

  WQ_v = Water Quality Volume, in watershed inches

For this example, the steps are as follows:

Compute modified CN for 1.5" rainfall

P = 1.5"

$Q = WQ_v \div \text{area} = (8{,}102\ ft^3 \div 3\ ac \div 43{,}560\ ft^2/ac \times 12\ in/ft) = 0.74"$

$CN = 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{1/2}]$

$= 1000/[10+5*1.5+10*0.74-10(0.74^2+1.25*0.74*1.5)^{1/2}]$

$= 95.01$

Use CN = 95

For CN = 95 and the T_c = 0.16 hours, compute the Q_p for a 1.5" storm. With the CN = 95, a 1.5" storm will produce 0.74" of runoff. I_a = 0.105, therefore I_a/P = 0.105/1.5 = 0.088. From Section 2.1, q_u = 625 csm/in, and therefore Q_{wq} = (900 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = 3.1 cfs.

Step 5 -- Size flow diversion structure (see Figure 6):

Size a low flow orifice to pass 3.1 cfs with 1.5' of head using the Orifice equation.

$Q = CA(2gh)^{1/2}$; 3.1 cfs = (0.6) (A) $[(2)\ (32.2\ ft/s^2)\ (1.5')]^{1/2}$

A = 0.53 sq ft = $\pi d^2/4$: d = 0.8' or 9.8"; use 10 inches

Size the 100-year overflow as follows: the 100-year wsel is set at 23.0. Use a concrete weir to pass the 100-year flow (39.0 cfs) into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

$Q = CLH^{3/2}$

$39 = 3.1\ (L)\ (2')^{1.5}$

L = 4.45'; use L = 4'-5" which sets flow diversion chamber dimension.

Weir wall elev. = 21.0. Set low flow invert at 21.0 - [1.5' + (0.5*10"*1ft/12")] = 19.08.

Step 6 -- Size filtration bed chamber (see Figure 7):

From Darcy's Law: $A_f = WQ_v\ (d_f) / [k\ (h_f + d_f)\ (t_f)]$
where d_f = 18"
k = 3.5 ft/day
h_f = 2.17'
t_f = 40 hours
A_f = (8,102 cubic feet) (1.5') / [3.5 (2.17' + 1.5') (40hr/(24hr/day))]

A_f = 567.7 sq ft; using a 2:1 ratio, say filter is 17' by 34' (= 578 sq ft)

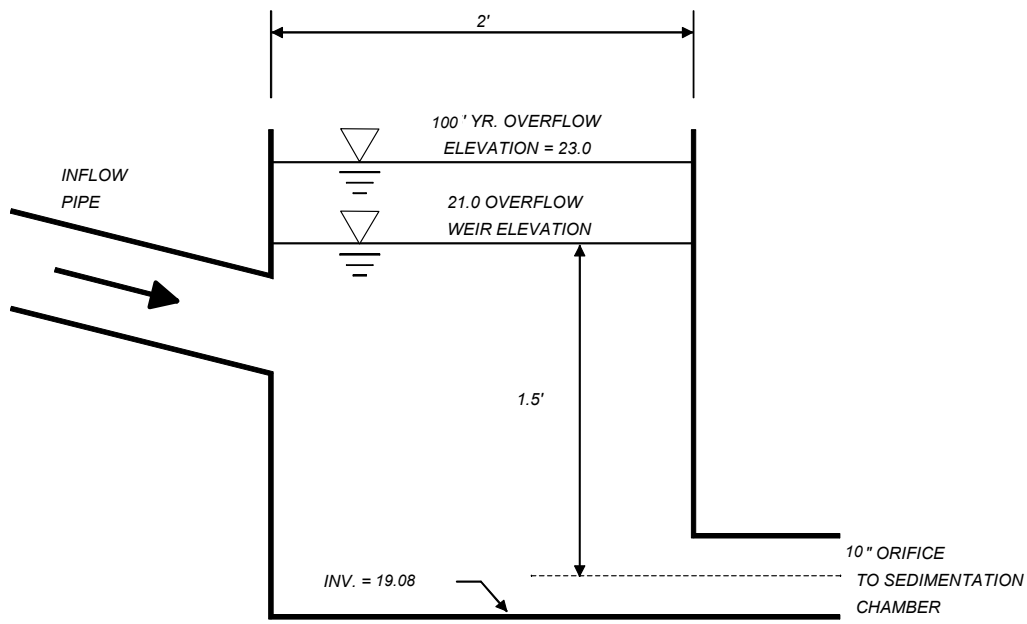

Figure 6. Flow Diversion Structure

Step 7-- Size sedimentation chamber

From Camp-Hazen equation, for I < 75%: $A_s = 0.066\ (WQ_v)$

A_s = 0.066 (8,102 cubic ft) or 535 sq ft

given a width of 17 feet, the length will be 535'/17' or 31.5 feet (use 17' x 32')

Step 8 -- Compute V_{min}

V_{min} = ¾(WQ_v) or 0.75 (8,102 cubic feet) = 6,077 cubic feet

Step 9 -- Compute storage volumes within entire facility and sedimentation chamber orifice size:

Volume within filter bed (V_f): $V_f = A_f\ (d_f)\ (n)$; n = 0.4 for sand
V_f = (578 sq ft) (1.5') (0.4) = 347 cubic feet

Temporary storage above filter bed ($V_{f\text{-}temp}$): $V_{f\text{-}temp} = 2\ h_f\ A_f$
$V_{f\text{-}temp}$ = 2 (2.17') (578 sq ft) = 2,509 cubic feet

Compute remaining volume for sedimentation chamber (V_s):
$V_s = V_{min} - [\ V_f + V_{f\text{-}temp}]$ or 6,077 - [347 + 2,509] = 3,221 cubic feet

Compute height in sedimentation chamber (h_s): $h_s = V_s/A_s$

(3,221 cubic ft)/(17' x 32') = 5.9' which is larger than the head available (4.33'); increase the size of the settling chamber, using 4.33' as the design height;

(3,221 cubic ft)/4.33' = 744 sq ft; 744'/17' yields a length of 43.8 feet (say 44')

New sedimentation chamber dimensions are 17' by 44'

With adequate preparation of the bottom of the settling chamber (rototil earth, place gravel, then surge stone), the bottom can infiltrate water into the substrate. The runoff will enter the groundwater directly without treatment. The stone will eventually clog without protection from settling solids, so use a removable geotextile to facilitate maintenance. Note that there is 2.17' of freeboard between bottom of recharge filter and water table.

Provide perforated standpipe with orifice sized to release volume (within sedimentation basin) over a 24 hr period (see Figure 8). Average release rate equals 3,221 ft^3/24 hr = 0.04 cfs

Equivalent orifice size can be calculated using orifice equation:

$Q = CA(2gh)^{1/2}$, where h is average head, or 4.33'/2 = 2.17'.
$0.04 \text{ cfs} = 0.6*A*(2*32.2 \text{ ft/s}^2*2.17 \text{ ft})^{1/2}$
$A = 0.005 \text{ ft}^2 = \pi D^2/4$: therefore equivalent orifice diameter equals 1".

Recommended design is to cap stand pipe with low flow orifice sized for 24 hr detention. Over-perforate pipe by a safety factor of 10 to account for clogging. Note that the size and number of perforations will depend on the release rate needed to achieve 24 hr detention. A multiple orifice stage-discharge relation needs to be developed for the proposed perforation configuration. Stand pipe should discharge into a flow distribution chamber prior to filter bed. Distribution chamber should be between 2 and 4 feet in length and same width as filter bed. Flow distribution to the filter bed can be achieved either with a weir or multiple orifices at constant elevation. See Figure 9 for stand pipe details.

Step 10 -- Design inlets, pretreatment facilities, underdrain system, and outlet structures

Step 11. Compute overflow weir sizes

Assume overflow that needs to be handled is equivalent to the 10" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 100-year peak discharge).

$Q = CA(2gh)^{½}$
$Q = 0.6(0.55 \text{ ft}^2)[(2)(32.2 \text{ ft/s}^2)(3.5 \text{ ft})]^{½}$
Q = 4.91 cfs, say 5.0 cfs

For the overflow from the sediment chamber to the filter bed, size to pass 5 cfs.

Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'
$5.0 = 3.1 * L * (0.5 \text{ ft})^{3/2}$
L = 4.56 ft, Use L = 4.75 ft.

Similarly, for the overflow from the filtration chamber to the outlet of the facility, size to pass 5.0 cfs.

Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'
$5.0 = 3.1 * L * (0.5 \text{ ft})^{3/2}$
L = 4.56 ft, Use L = 4.75 ft.

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

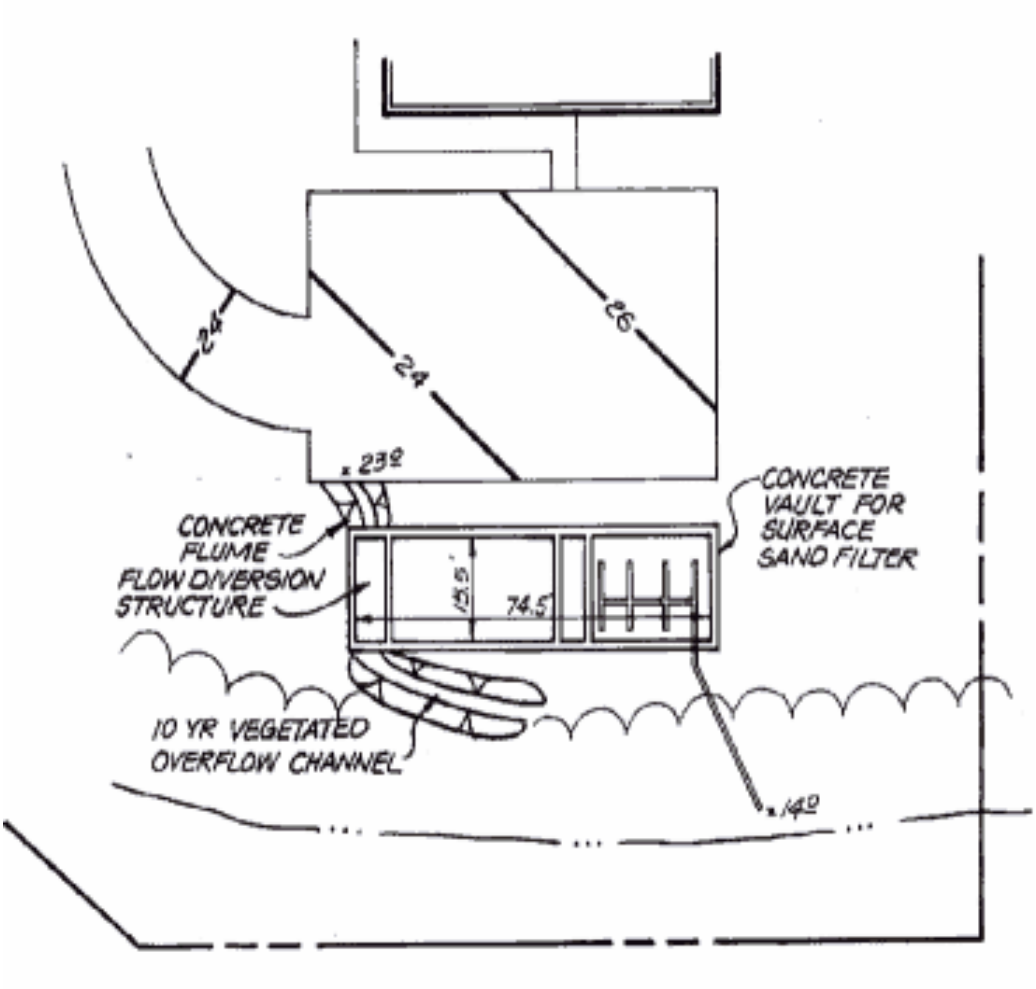

Figure 7. Surface Sand Filter Site Plan

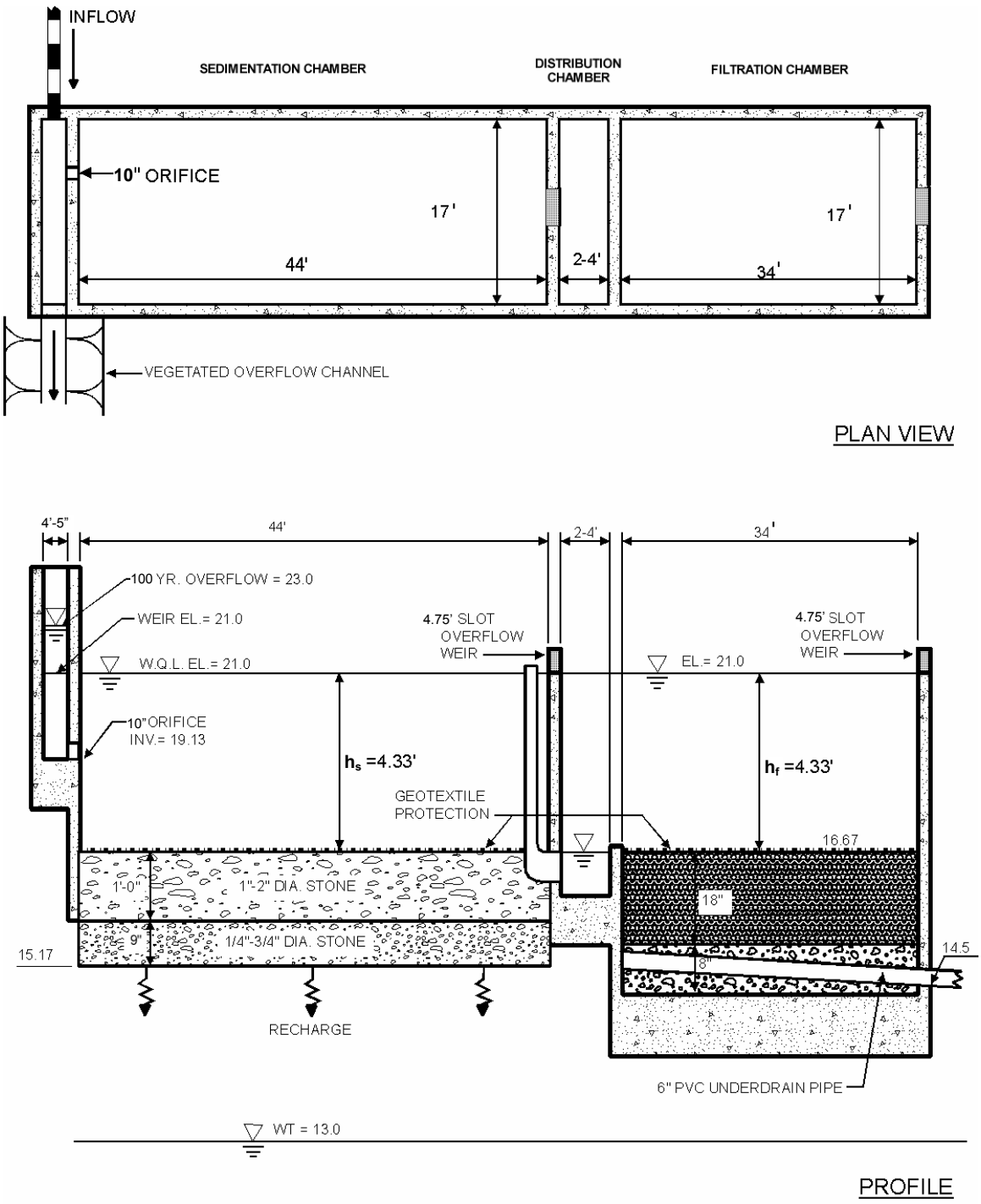

Figure 8. Plan and Profile of Surface Sand Filter

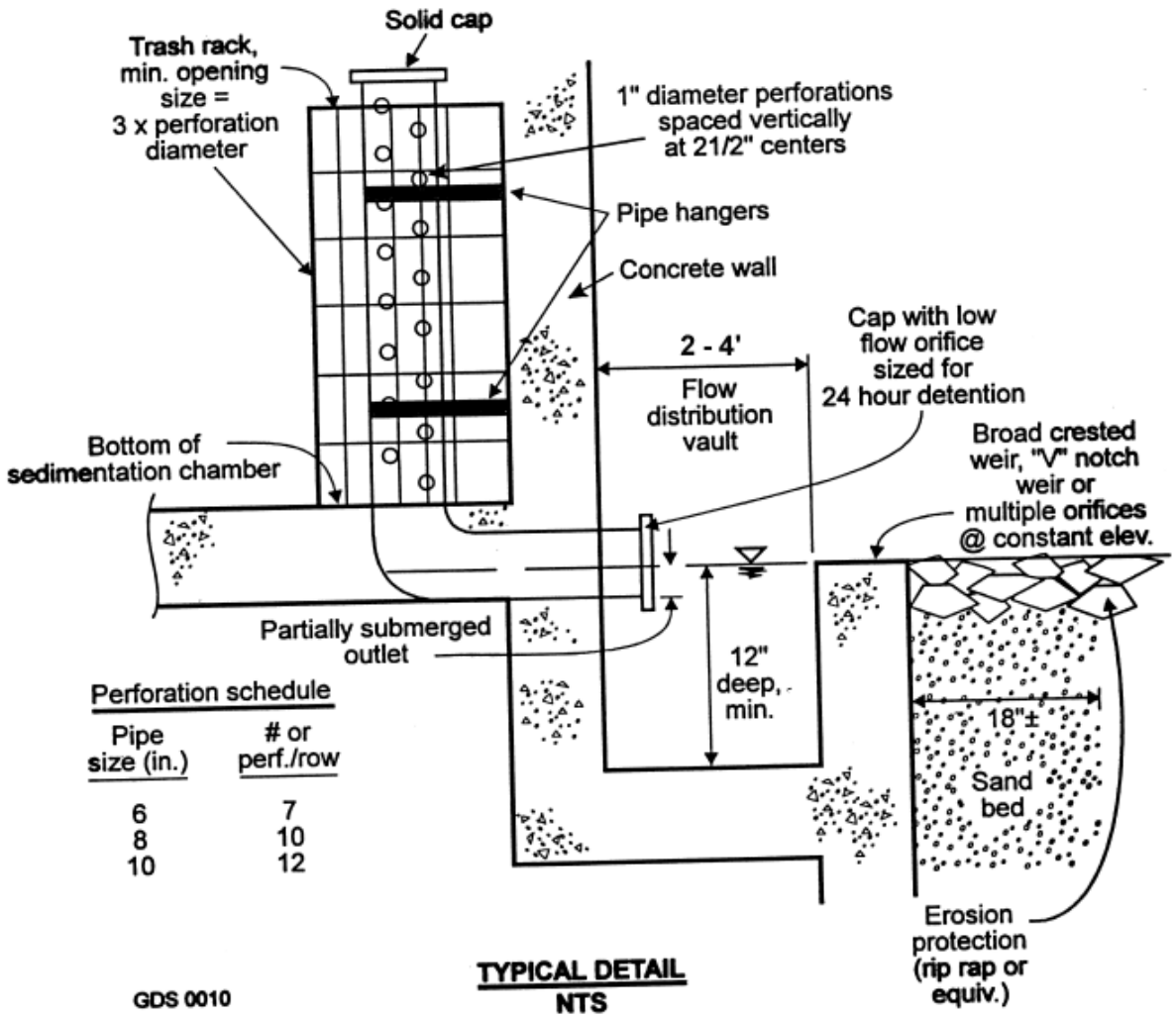

Figure 9. Perforated Stand Pipe Detail

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D.4 Infiltration Trench Design Example

This example focuses on the design of an infiltration trench to meet the water quality treatment requirements of the site. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of infiltration trenches is to provide water quality treatment and groundwater recharge and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Cottonwood Creek Community Center is shown in Figure 1.

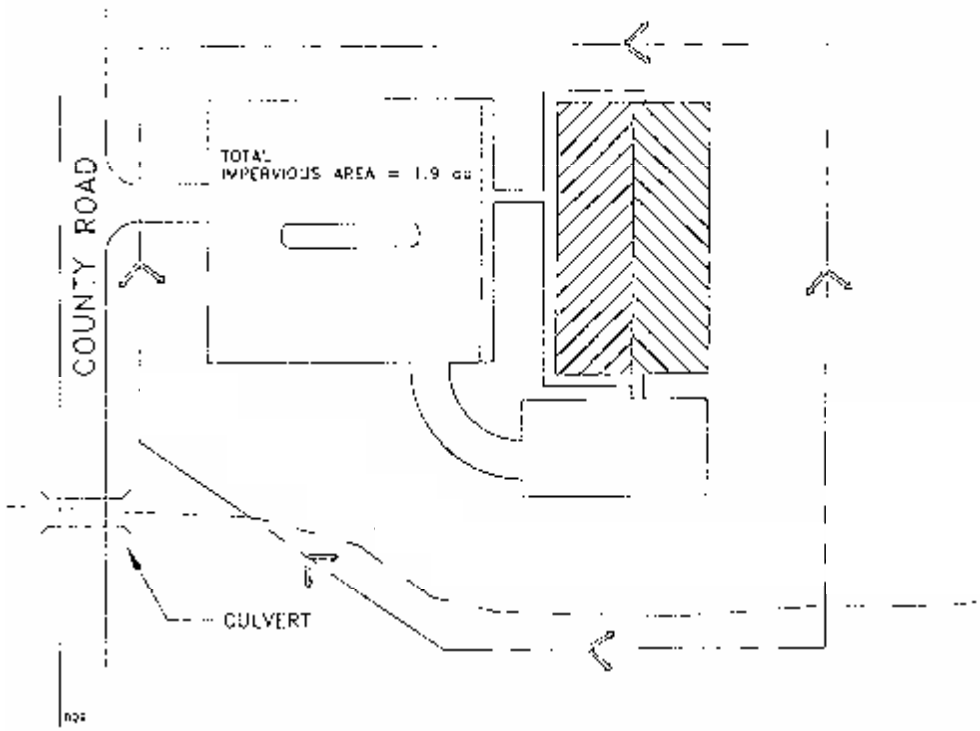


Figure 1. Cottonwood Creek Community Center Site Plan

Base Data		Hydrologic Data	
Site Area = Total Drainage Area (A) = 3.0 ac		Pre	Post
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	55	82
Soils Type "B" Wise County	t_c	0.42 hr	0.16 hr

Step 1 -- Compute runoff control volumes from the *integrated* Design Approach

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  R_v = 0.05 + (63.3) (0.009) = 0.62

- Compute WQ_v

  WQ_v = (1.5") (R_v) (A) / 12
  = (1.5") (0.62) (3.0 ac) (43,560 ft^2/ac) (1ft/12in)
  = 8,102 ft^3 = 0.186 ac-ft

Compute Stream Streambank Protection Volume, (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

- Develop Site Hydrologic and Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations

  Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\text{ year}}$
		Inches	*cfs*	*cfs*
Pre-developed	55	0.11	0.1	8.9
Post-Developed	82	1.1	4.4	29.0

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (I_a) for CN of 82 is 0.44: (TR-55) [I_a = (200/CN - 2)]

  I_a/P = (0.44)/ 2.64 inches = 0.17
  T_c = 0.17 hours
  q_u = 850 csm/in

Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.02

$V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

V_s/V_r = 0.655

Therefore, $V_s = SP_v$ = 0.655(1.10")(1/12)(3 ac) = 0.18 ac-ft = 7,841 ft^3

- Define the average extended detention Release Rate

  The above volume, 0.18 ac-ft, is to be released over 24 hours.
  (0.18 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 0.09 cfs

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure.

Table 1 Summary of General Design Information for Falcon Creek Community Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	7,841	
Q_f	Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	Cottonwood Creek			EWB
DRAINAGE AREA NAME:	Pre-Developed Conditions			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
meadow (good cond.)	B		55	3.00
			AREA SUBTOTALS:	3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.36"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	dense grass	'n'= 0.24	150ft	1.50%
				0.36 hrs
Shallow Flow	unpaved		500 ft	2.00%
			2.28 fps	0.06 hrs
Channel Flow				
Total Area in Acres =	3.00	Total Sheet Flow = 0.36 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	55			
Time of Concentration =	0.42 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.11	0.10
2 Year	3.36	0.30	0.39
5 Year	4.56	0.77	1.5
10 Year	5.28	1.12	2.5
25 Year	6.72	1.95	4.8
50 Year	7.68	2.57	6.5
100 Year	8.88	3.40	8.9

Figure 2. Cottonwood Creek Community Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Cottonwood Creek			EWB
DRAINAGE AREA NAME:	Post-Developed Conditions			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
meadow (good cond.)	B		55	1.10
impervious	B		98	1.90
			AREA SUBTOTALS:	38.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.36"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	short grass	'n'= 0.15	50 ft	1.50%
				0.10 hrs
Shallow Flow	paved		600 ft	2.00%
			2.88 fps	0.06 hrs
Channel Flow		'n'= 0.24	50 ft	2.00%
Hydraulic Radius = 0.75	X-S estimated	WP estimated	7.25 fps	0.00 hrs
Total Area in Acres =	3.00	Total Sheet Flow = 0.10 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	82			
Time of Concentration =	0.16 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.10	4.4
2 Year	3.36	1.67	6.8
5 Year	4.56	2.69	11
10 Year	5.28	3.33	14
25 Year	6.72	4.65	20
50 Year	7.68	5.56	24
100 Year	8.88	6.70	29

Figure 3. Cottonwood Creek Community Center Post-Developed Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of an infiltration trench

Site Specific Data:

Table 2 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench.

Table 2 Site Specific Data

Criteria	Value
Soil	Sandy Loam
Percolation Rate	1"/hour
Ground Elevation at BMP	20'
Seasonally High Water Table	13'
Stream Invert	12'
Soil slopes	<1%

Step 3 -- Confirm local design criteria and applicability

Table 3, below, summarizes the requirements that need to be met to successfully implement infiltration practices. On this site, infiltration is feasible, with restrictions on the depth and width of the trench.

Table 3 Infiltration Feasibility

Criteria	Status
Infiltration rate (f_c) greater than or equal to 0.5 inches/hour.	Infiltration rate is 1.0 inches/hour. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	Sandy Loam meets both criteria.
Infiltration cannot be located on slopes greater than 6% or in fill soils.	Slope is <1%; not fill soils. OK.
Hotspot runoff should not be infiltrated.	Not a hotspot land use. OK.
Infiltration is prohibited in karst topography.	Not in karst. OK.
The bottom of the infiltration facility must be separated by at least two feet vertically from the seasonally high water table.	Elevation of seasonally high water table: 13' Elevation of BMP location: 20'. The difference is 7'. Thus, the trench can be up to 5' deep. OK.
Infiltration facilities must be located 100 feet horizontally from any water supply well.	No water supply wells nearby. OK.
Maximum contributing area generally less than 5 acres. (Optional)	3 acres. OK.
Setback 25 feet down-gradient from structures.	Fifty feet straight-line distance between the parking lot and the tree line. OK if the trench is 25' wide or narrower.

Step 4 -- Compute WQ_v peak discharge (Q_{wq})

- Compute Water Quality Volume:

  WQ_v previously determined to be 8,102 cubic feet.

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. Conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure or leads to the design of undersized grass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

$$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\ QP)^{1/2}]}$$

where P = rainfall, in inches (use 1.5" for the Water Quality Storm)
and Q = runoff volume, in inches (equal to $WQ_V \div$ area)

- Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55, Chapter 3: "Time of concentration and travel time").
- Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the Water Quality Storm is computed (based on the procedures identified in TR-55, Chapter 4: "Graphical Peak Discharge Method"). Use appropriate rainfall distribution type (typically Type II in the North Central Texas region).
  - Read initial abstraction (I_a), compute I_a/P
  - Read the unit peak discharge (q_u) for appropriate t_c
  - Using the water quality volume (WQ_v), compute the peak discharge (Q_{wq})

$$\mathbf{Q_{wq} = q_u{*}A{*}WQ_V}$$

where
Q_{wq}= the peak discharge, in cfs
q_u = the unit peak discharge, in cfs/mi²/inch
A = drainage area, in square miles
WQ_v = Water Quality Volume, in watershed inches

For this example, the steps are as follows:

Compute modified CN for 1.5" rainfall
P = 1.5"
Q = $WQ_v \div$ area = (8,102 $ft^3 \div$ 3 ac $\div$ 43,560 ft^2/ac $\times$ 12 in/ft) = 0.74"

CN = $1000/[10 + 5P + 10Q - 10\ (Q^2 + 1.25*Q*P)^{1/2}]$
= $1000/[10 + 5*1.5 + 10*0.74 - 10(0.74^2 + 1.25*0.74*1.5)^{1/2}]$
= 91.1
Use CN = 91

For CN = 91 and the T_c = 0.16 hours, compute the Q_{wq} for a 1.5" storm. With the CN = 91, a 1.5" storm will produce 0.74" of runoff. I_a = 0.198, therefore I_a/P = 0.198/1.5 = 0.132. q_u = 825 csm/in, and

therefore:

Q_{wq} = (825 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = 2.86 cfs.

Step 5 -- Size the infiltration trench

The area of the trench can be determined by the following equation:

$$A = \frac{WQ_v}{(nd + kT/12)}$$

Where:

A = Surface Area
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T= Fill Time (time for the practice to fill with water), in hours

Assume that:

n = 0.32
d = 5 feet (see above; feasibility criteria)
k = 1 inch/hour (see above; site data)
T = 2 hours

Therefore:

A = 8,102 ft^3 / (0.32 × 5 + 1 × 2/12)ft
A = 4,586 ft^2

Since the width can be no greater than 25' (see above; feasibility), determine the length:

L = 4,586 ft^2 / 25 ft
L = 183 feet

Assume that 1/3 of the runoff from the site drains to Point A and 2/3 drains to Point B. Use an L-shaped trench in the corner of the site (see Figure 4 for a site plan view). The surface area of the trench is proportional to the amount of runoff it drains (e.g., the portion draining from Point A is half as large as the portion draining Point B).

Step 6 -- Size the flow diversion structures

Since two entrances are used, two flow diversions are needed.

For the entire site:

$Q_{100\text{-year}}$ = 29 cfs (See Figure 3)
Peak flow for WQ_v = 2.86 cfs. (Step 3).

For the first diversion (Point A)

Assume peak flow equals 1/3 of the value for the entire site.
Thus, $Q_{100\text{-year}}$ = 29/3 = 9.7 cfs
Peak flow for WQ_v = 2.86/3 = 0.95 cfs

Size the low flow orifice to pass 0.95 cfs with 1.5' of head using the Orifice equation.

$Q=CA(2gh)^{½}$; 0.95 cfs = $0.6A(2 \times 32.2\ ft/s^2 \times 1.5')^{½}$
A=0.16 sq. ft. = $\pi d^2/4$; d = 0.45'; use 6" pipe with 6" gate valve

Size the 100-year overflow weir crest at 22.5'. Use a concrete weir to pass the 100-year flow (9.7 – 0.95 = 8.75 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L = Q/(CH^{1.5})$
$L = 8.75$ cfs/ $(3.1)(1)^{1.5} = 2.8'$; <u>use 2.8'</u> (see Figure 5)

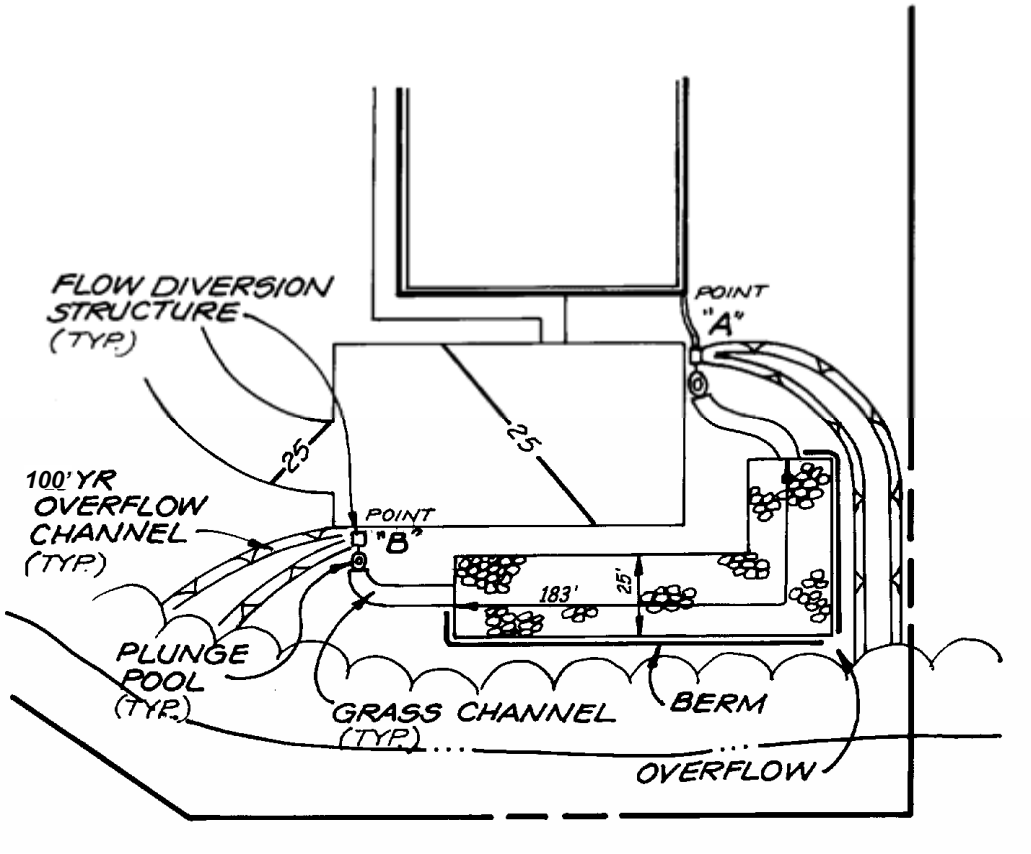


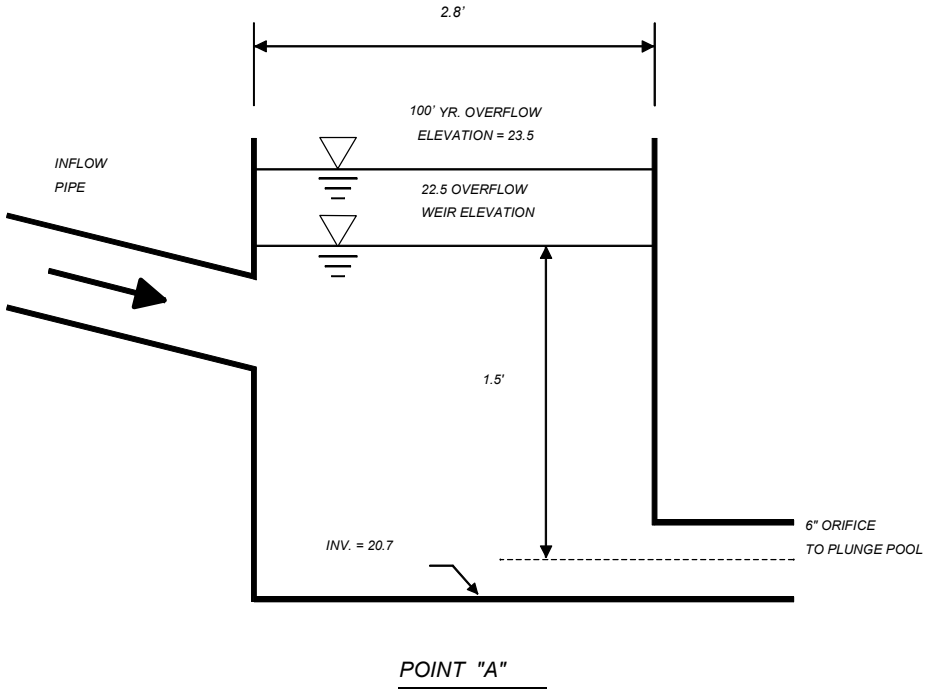
Figure 4. Infiltration Trench Site Plan

Size the second diversion (Point B) using the same techniques.
Peak flow equal 2/3 of the value for the entire site. Thus:
$Q_{100\text{-year}} = 29*0.67 = 19.3$ cfs
Peak flow for $WQ_v = 2.86*0.67 = 1.47$ cfs

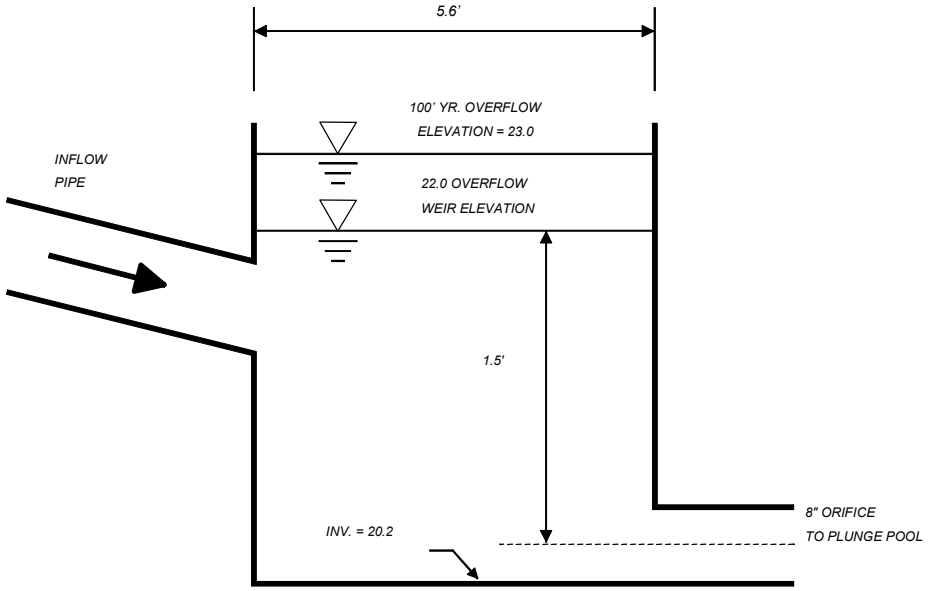
Size the low flow orifice to pass 1.47 cfs with 1.5' of head using the Orifice equation.
$Q=CA(2gh)^{½}$; 1.91 cfs = $0.6A(2 \times 32.2 \text{ ft/s}^2 \times 1.5')^{½}$
A=0.32 sq. ft. = $\pi d^2/4$; d = 0.64'; use 8" pipe with 8" gate valve

Size the 100-year overflow weir crest at 22.0'. Use a concrete weir to pass the 100-year flow (19.3 – 1.91 = 17.39 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L = Q/(CH^{1.5})$
$L = 17.39$ cfs/ $(3.1)(1)^{1.5} = 5.6$; <u>use 5.6'</u> (see Figure 5)



POINT "A"



POINT "B"

Figure 5. Flow Diversion Structures

Step 7. Size pretreatment volume and design pretreatment measures

As rule of thumb, size pretreatment to treat 25% of the WQ_v. Therefore, treat 8,102 × 0.25 = 2,026 ft^3.

For pretreatment, use a pea gravel filter layer with filter fabric, a plunge pool, and a grass channel.

Pea Gravel Filter

The pea gravel filter layer covers the entire trench with 2" (see Figure 6). Assuming a porosity of 0.32, the water quality treatment in the pea gravel filter layer is:
WQ_{filter}= (0.32)(2")(1 ft/12 inches)(4,586 ft^2) = 245 ft^3

Plunge Pools

Use a 5'X10' plunge pool at Point A and a 10'X10' plunge pool at Point B with average depths of 2'.

Total WQ_{pool}= (10 ft)(10+5 ft)(2 ft) = 300 ft^3

Grass Channel

Thus, the grass channel needs to treat at least (2,026 - 245 - 300)ft^3 = 1,481 ft^3

Use a Manning's equation nomograph or software to size the swale.

The channel at point A should treat one third of 1,481 ft^3 or 494 ft^3

- Assume a trapezoidal channel with 4' channel bottom, 3H:1V side slopes, and a Manning's n value of 0.15. Use a nomograph to size the swale; assume a 1% slope.
- Use a peak discharge of 0.95 cfs (Peak flow for one third of WQ_v, or 2,700 ft^3)
- Compute velocity: V=0.47 ft/s

- To retain the 1/3 of the WQ_v (2,700 ft^3) for 10 minutes, the length would be 282 feet.
- Since the swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 494 ft^3, the length should be pro-rated to reflect this reduction.

  Therefore, adjust length:

  L= (282 ft)(494 ft^3/2,700 ft^3) =52 feet. <u>Use 55 feet.</u>

  The channel at point B should treat two thirds of 1,481 ft^3, or 988 ft^3

- Assume a trapezoidal channel with 5' channel bottom, 3H:1V side slopes, and a Manning's n value of 0.12. Use a nomograph to size the swale; assume a 0.5% slope.
- Use a peak discharge of 1.91 cfs (Peak flow for two thirds of WQ_v, or 5401 ft^3)
- Compute velocity: V=0.51 ft/s

- To retain the 2/3 of the WQ_v (5,401 ft^3) for 10 minutes, the length would be 306 feet.
- Since the swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 988 ft^3, the length should be pro-rated to reflect this reduction.

  Therefore, adjust length:

  L= (306 ft)(988 ft^3/5,401 ft^3) = 55 feet. <u>Use 55 feet.</u>

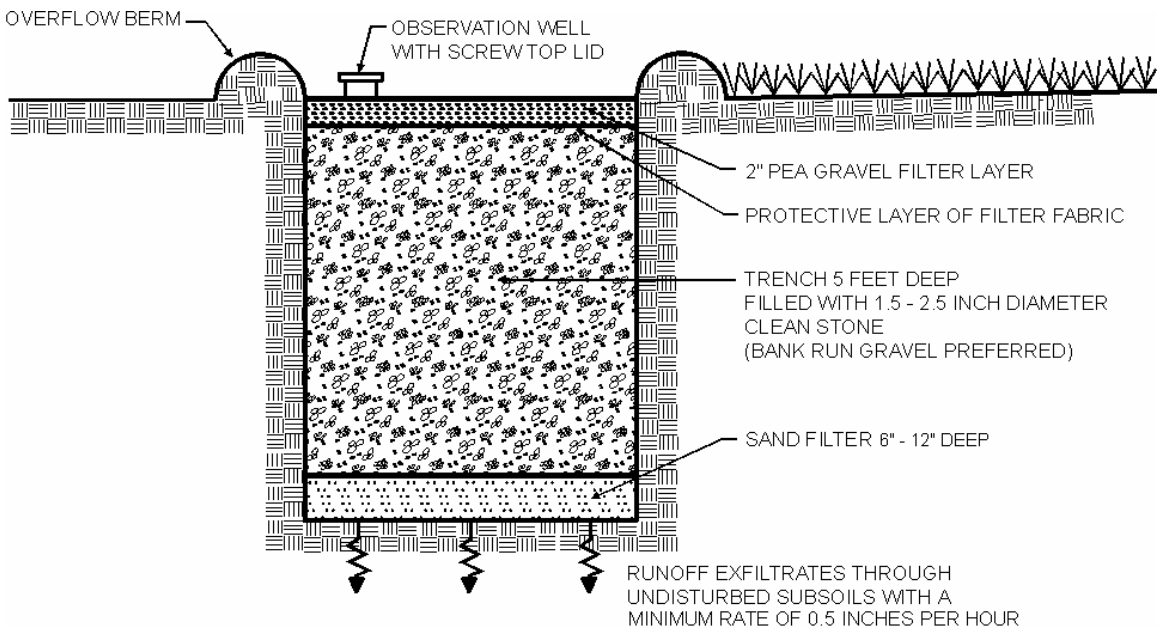


Figure 6. Infiltration Trench Cross Section

Step 8 -- Design Spillway(s)

Adequate storm water outfalls should be provided for the overflow associated with the 100-year and larger design storm events, ensuring non-erosive velocities on the down slope.

D.5 Enhanced Swale Design Example

This example focuses on the design of a dry swale to meet the water quality treatment requirements of the site. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). It is assumed that the Local Criteria requires enhanced swales to adequately convey the 25-year peak flow. Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of dry swales is to provide water quality treatment and groundwater recharge and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Wellington Recreation Center is shown in Figure 1.

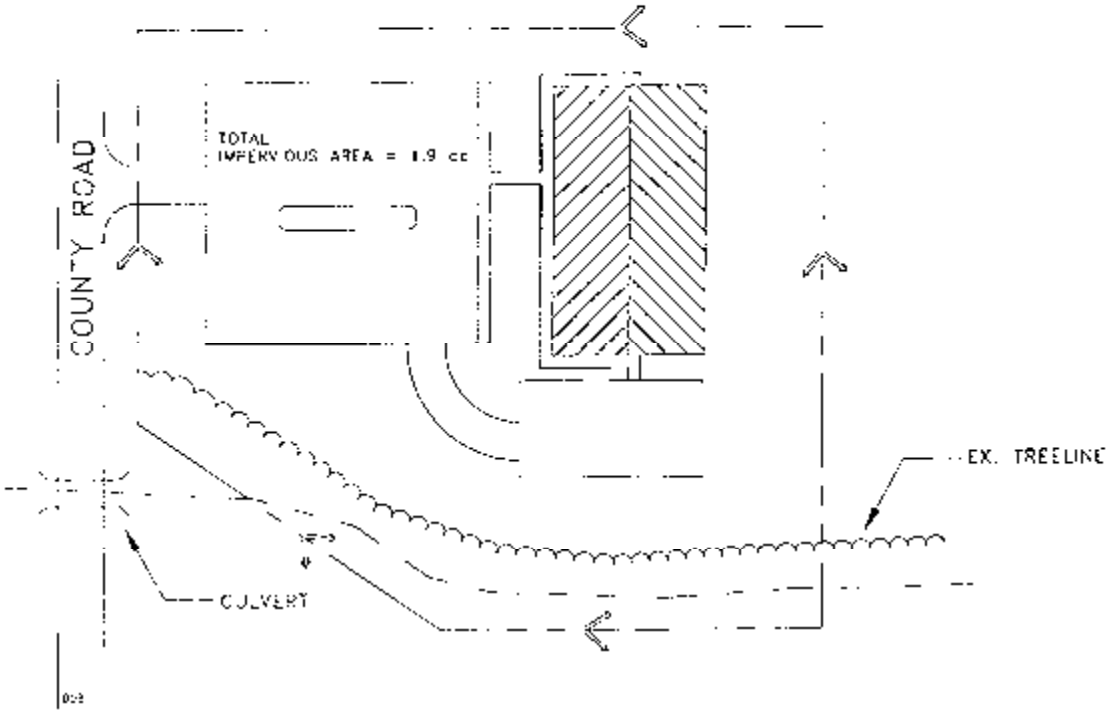


Figure 1. Wellington Recreation Center Site Plan

Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac		**Pre**	**Post**
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	77	91
Soils Type 50% “C”, 50% “D” Tarrant County	t_c	.41	.20

Computation of Preliminary Storm Water Storage Volumes and Peak Discharges

Two swales will be designed to carry flow to the existing stream, one around each side of the development.

Step 1 -- Compute runoff control volumes from the *integrated* Design Approach

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.5") (R_v) (A) / 12$
  $= (1.5") (0.62) (3.0ac) (43{,}560ft^2/ac) (1ft/12in)$
  $= 8{,}102\ ft^3 = 0.19$ ac-ft

Compute Stream Streambank Protection Volume (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for streambank protection and flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 2.1 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Per attached TR-55 calculations (Figures 2 and 3)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*
Pre-developed	74	0.69	1.7	17.0
Post-Developed	90	1.66	6.5	33.0

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (Ia) for CN of 90 is 0.222: [$I_a = (200/CN - 2)$]

  $I_a/P = (0.222)/ 2.64$ inches $= 0.08$
  $T_c = 0.21$ hours
  $q_u = 840$ csm/in

Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.022

For a Type II rainfall distribution,

$V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

$V_s/V_r = 0.65$

Therefore, $V_s = SP_v = 0.65(1.66")(1/12)(3\ ac) = 0.27$ ac-ft $= 11{,}761\ ft^3$

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure.

Table 1 Summary of General Design Information for Wellington Recreation Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	11,761	
Q_f	Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	Wellington of Recreation Center			EWB
DRAINAGE AREA NAME:	Pre-Developed Conditions			
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
woods (good cond.)	C		70	1.5
woods (good cond.)	D		78	1.5
			AREA SUBTOTALS:	3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.36"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	dense grass	'n'= 0.24	150 ft	1.50%
				0.36 hrs
Shallow Flow	unpaved		500 ft	2.00%
			2.28 fps	0.06 hrs
Channel Flow				
Total Area in Acres =	3.00	Total Sheet Flow = 0.36 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	74			
Time of Concentration =	0.42 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.69	1.7
2 Year	3.36	1.14	3.0
5 Year	4.56	2.02	5.4
10 Year	5.52	2.79	7.6
25 Year	6.72	3.80	11.0
50 Year	7.92	4.85	14.0
100 Year	9.12	5.94	17.0

Figure 2. Wellington Recreation Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Wellington on Recreation Center			EWB
DRAINAGE AREA NAME:	Post-Developed Conditions			3-Jan-00
COVER DESCRIPTION	SOIL GROUP A, B, C, D?	C from TABLE 2.1.4-2	CN from TABLE 2.1.5-1	AREA (in acres)
open space (good cond.)	C		74	0.25
woods (good cond.)	C		70	0.30
impervious	C		98	1.90
open space (good cond.)	D		80	0.25
woods (good cond.)	D			0.30
			AREA SUBTOTALS:	3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 3.36"	Cross Section	Wetted Per	Avg Velocity	Tt (hrs)
Sheet Flow	dense grass	'n'= 0.24	50 ft	1.50%
				0.15 hrs
Shallow Flow	paved		600 ft	2.00%
			2.87 fps	0.06 hrs
Channel Flow		'n'= 0.024	50 ft	2.00%
Hydraulic Radius= 0.75	X-S estimated	WP estimated	7.25 fps	0.00hrs
Total Area in Acres =	3.00	Total Sheet Flow = 0.15 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	90			
Time of Concentration =	0.21 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.66	6.5
2 Year	3.36	2.32	9.3
5 Year	4.56	3.45	14.0
10 Year	5.52	4.38	18.0
25 Year	6.72	5.55	23.0
50 Year	7.92	6.73	28.0
100 Year	9.12	7.91	33.0

Figure 3. Wellington Recreation Center Post-Developed Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of an enhanced dry swale system

Existing ground elevation at the facility location is 922.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 913.0 feet and underlying soils are predominately clay. Adjacent creek invert is at 912.0 feet.

Step 3 -- Confirm local design criteria and applicability

There is a local requirement that the 25-year storm is contained within the top of banks of all channels, including these enhanced swale controls.

No additional local criteria are applicable.

Step 4 -- Determine pretreatment volume

Size two shallow forebays at the head of the swales equal to 0.05" per impervious acre of drainage (each) (Note, total recommended pretreatment requirement is 0.1"/imp acre). (1.9 ac) (0.05") (1ft/12") (43,560 sq ft/ac) = 344.9 ft^3

Use a 2' deep pea gravel drain at the head of the swale to provide erosion protection and to assist in the distribution of the inflow. There will be no side inflow nor need for pea gravel diaphragm along the sides.

Step 5 -- Determine swale dimensions

Required: bottom width, depth, length, and slope necessary to store WQ_v with less than 18" of ponding (see Figure 5 for representative site plan).

Unit Peak Discharge (q_u), csm/in

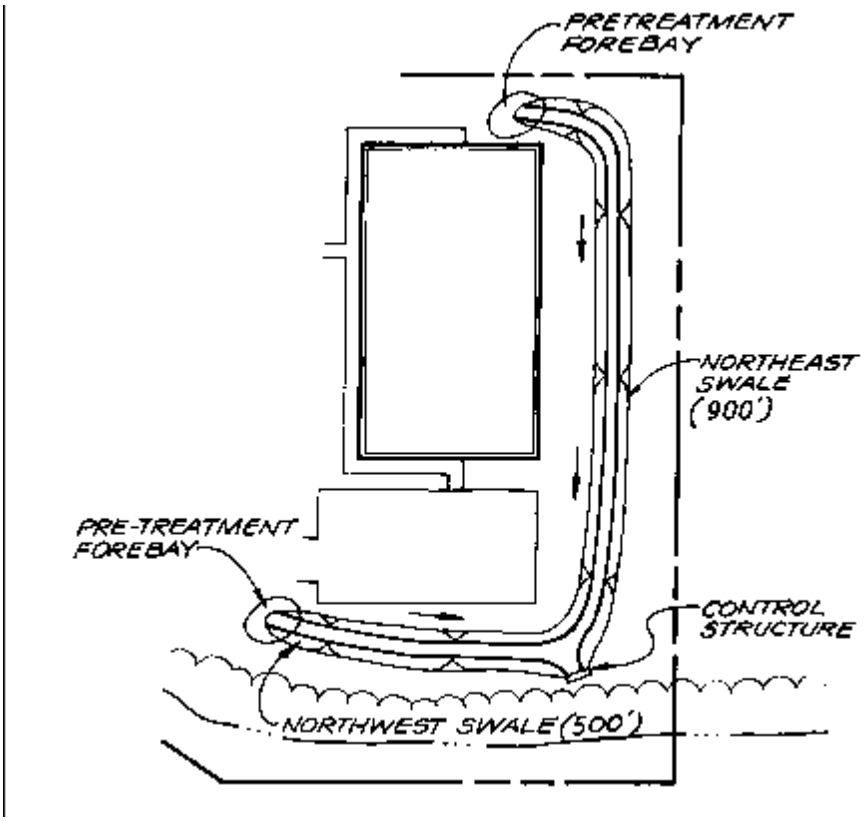


Figure 5. Enhanced Dry Swale Site Plan

Assume a trapezoidal channel with a maximum WQ_v depth of 18". Control for this swale will be a shallow concrete wall with a low flow orifice, trash rack located per Figures 5 and 6. Per the site plan, we have about 1,400' of swale available, if the swale is put in with two tails. The outlet control will be set at the existing invert minus three feet (922.0 - 3.0 = 919.0). The existing uphill invert for the northwest fork is 924.0 (length of 500'), the invert for the northeast fork is 928.0 (at a length of 900').

Slope of northwest fork is (924 - 919)/500' = 0.01 or 1.0%
Slope of northeast fork is (928 - 919)/900' = 0.01 or 1.0%

Minimum slope is 1.0 % [okay]

For a trapezoidal section with a bottom width of 6', a WQ_v average depth of 9", 3:1 side slopes, compute a cross sectional area of (6') (0.75') + (0.75') (2.25') = 6.2 ft^2 (see Figure 7).

(6.2 sq ft) (1,400 ft) = 8,680cubic feet [> WQ_v of 8,102 ft^3; OK]

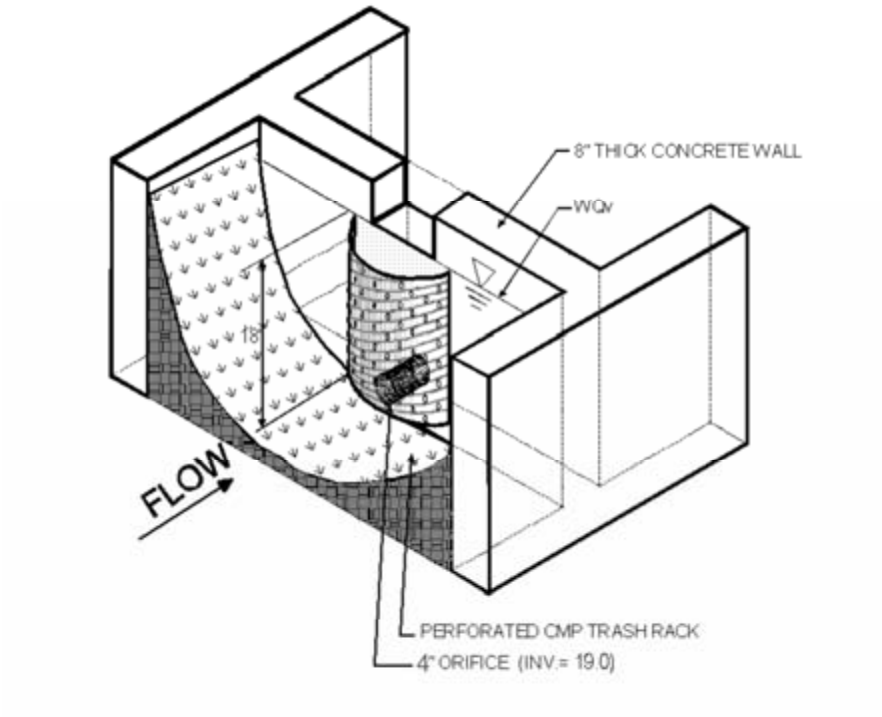


Figure 6 Control Structure at End of Swale

Step 6 -- Compute number of check dams (or similar structure) required to detain WQ_v (see Figure 7)

For the northwest fork, 500 ft @ 1.0% slope, and maximum depth at 18”, place checkdams at:
1.5’/0.01 = 150' place at 150’, 4 required

For the northeast fork, 900 ft @ 1.0% slope, and maximum 18" depth, place checkdams at:
1.5’/0.01 = 150’ place at 150’, 6 required

Step 7 -- Calculate draw-down time

In order to ensure that the swale will draw down within 24 hours, the planting soil will need to pass a maximum rate of 1.5’ in 24 hours (k = 1.5’ per day). Provide 6” perforated underdrain pipe and gravel system below soil bed (see Figure 7).

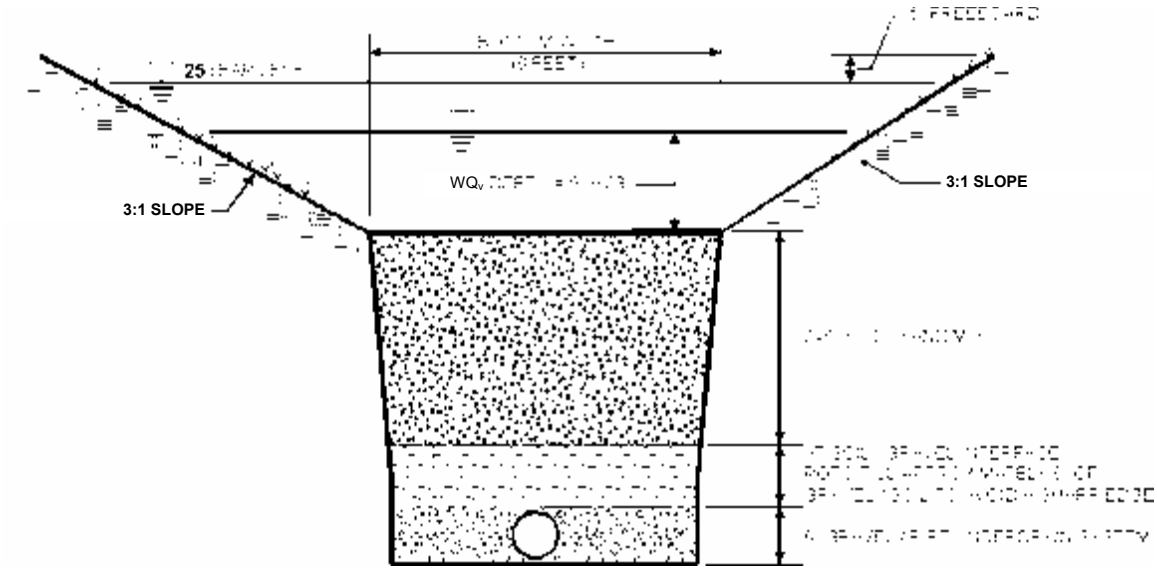


Figure 7 Trapezoidal Dry Swale Section

Step 8 -- Check 25-year flows for velocity erosion potential and freeboard

Given the local requirements to contain the 25-year flow within banks with freeboard. In this example only the 25-year flow will be checked assuming that lower flows will be handled. The 25-year flow is 23.0 cfs, assume that 30% goes through northwestern swale (6.9 cfs) and 70% goes through the northeastern swale (9.3 cfs). Design for the larger amount (13.3 cfs). From separate computer analysis, with a slope of 1.0%, the 25-year velocity will be 2.7 feet-per-second at a depth of .63 feet, provide an additional .5' of freeboard above top of checkdams or about 1.2' (total channel depth = 2.7').

Find 25-year overflow weir length required: (weir eq. $Q= CLH^{3/2}$), where C = 3.1, Q_{25} = 23 cfs, H =1.2; Rearranging the equation yields:

$L = 23 \text{ cfs}/ (3.1*1.2^{1.5}) = 5.6'$ <u>Use 5 ft</u>

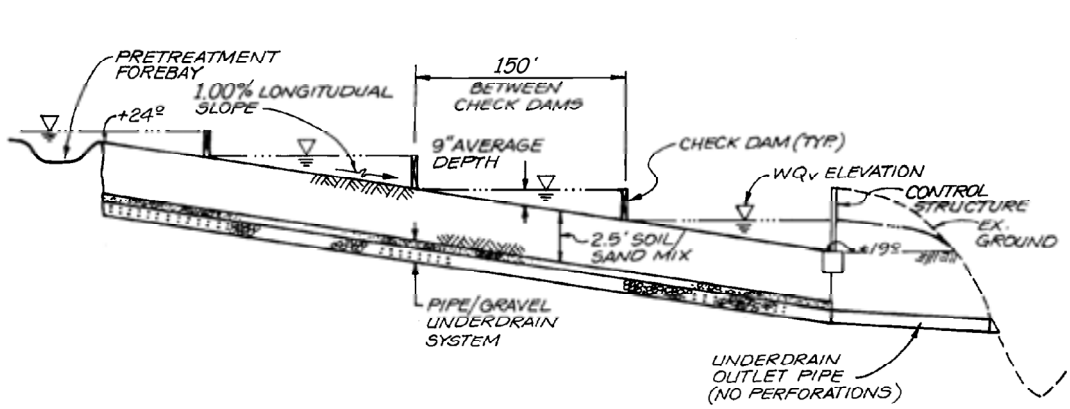


Figure 8 Profile of Northwest Fork Dry Swale

Step 9 -- Design low flow orifice at downstream headwall and checkdams (See Figure 6)

Design orifice to pass 8,102 cubic feet in 6 hours.

8,102 cubic feet/ [(6 hours) (3600 sec/hour)] = 0.4 cfs
Use Orifice equation: $Q = CA(2gh)^{1/2}$

Assume h = 1.5'

$A = (0.4 \text{ cfs}) / [(0.6) ((2) (32.2 \text{ ft/s}^2) (1.5'))^{1/2}]$

A = 0.068 sq ft, dia = 0.29 feet or 3.6" <u>Use 4" orifice.</u>

Provide 3" v-notch slot in each check dam.

Step 10 – Design inlets, sediment forebay(s), and underdrain system

See Figure 8

Step 11 – Prepare Vegetation and Landscaping Plan

APPENDIX

E

iSWM™ WORKSHEETS AND CHECKLISTS

Table of Contents

Worksheet for Conceptual iSWM Site Plan

Checklist for Conceptual iSWM Site Plan Preparation and Review

Checklist for Preliminary iSWM Site Plan Preparation and Review

Checklist for Final iSWM Site Plan Preparation and Review

Storm Water Pond Maintenance Checklist

Filtration Facility Maintenance Checklist

Infiltration Trench Maintenance Checklist

Enhanced Swale/Grass Channel/Filter Strip Maintenance Checklist

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Worksheet for Conceptual iSWM Site Plan

	Included? Yes	No	Comments
I. Review of *integrated* Site Design Practices			
A. Conservation of Natural Features and Resources			
Are there opportunities to:			
1. conserve undisturbed vegetation?	____	____	_____
2. conserve floodplains?	____	____	_____
3. conserve steep slopes?	____	____	_____
4. conserve natural drainageways?	____	____	_____
5. conserve streams?	____	____	_____
6. conserve wetlands?	____	____	_____
7. conserve other natural features?	____	____	_____
8. conserve critical areas?	____	____	_____
B. Lower Impact Site Design Techniques			
Are there opportunities to:			
1. reduce clearing and grading?	____	____	_____
2. locate development in less sensitive areas?	____	____	_____
3. utilize open space development?	____	____	_____
4. utilize other techniques?	____	____	_____
C. Reduction of Impervious Cover			
Are there opportunities to:			
1. reduce roadway lengths?	____	____	_____
2. reduce roadway widths?	____	____	_____
3. reduce building footprints?	____	____	_____
4. reduce parking lots?	____	____	_____
5. utilize fewer or alternative cul-de-sacs?	____	____	_____
6. utilize pervious pavement?	____	____	_____
7. utilize other reduction measures?	____	____	_____
D. Utilization of Natural Features for Storm Water Management			
Are there opportunities to:			
1. utilize buffers?	____	____	_____
2. utilize undisturbed areas?	____	____	_____
3. utilize natural drainageways vs storm drain systems?	____	____	_____
4. utilize vegetated swales vs curb and gutter?	____	____	_____
5. drain runoff to pervious areas?	____	____	_____
6. utilize other measures or features?	____	____	_____
E. Can Credits be obtained for use of *integrated* Site Design Practices?			

Worksheet for Conceptual iSWM Site Plan (Continued)

	Comments
2. Review ***integrated* Design Approach**	
A. How will the *integrated* Design Approach be addressed for **Water Quality Protection?**	
B. How will the *integrated* Design Approach be addressed for **Streambank Protection?**	
C. How will the *integrated* Design Approach be addressed for **On-Site Flood Control?**	
D. How will the *integrated* Design Approach be addressed for **Downstream Flood Control?**	

	Yes	No	Comments
3. Review of **Water Quality Volume Reduction Methods**			
Can the Water Quality Volume be reduced by:			
A. **conservation and/or restoration of natural areas?**			
B. use of **stream buffers?**			
C. use of **vegetated channels?**			
D. use of **overland flow filtration/groundwater recharge?**			
E. use of **low imperviousness development?**			

	Yes	No	Comments
4. Review of potential **hot spots**			
Are any of the following located or to be located on the site which may require special treatment and design consideration?			
A. Gas/fueling stations?			
B. Vehicle maintenance areas?			
C. Vehicle washing/steam cleaning areas?			
D. Auto recycling facilities?			
E. Illegal dumpsites?			
F. Outdoor material storage areas?			
G. Loading and transfer areas?			
H. Landfills?			
I. Industrial facilities?			
J. Other areas that may be a potential pollution source?			

Worksheet for Conceptual iSWM Site Plan (Continued)

	Included?		
	Yes	No	Comments
5. Review of ***integrated* Storm Water Controls**			
Which of the following controls will potentially be used on the site?			
A. Alum Treatment System?	____	____	_____
B. Bioretention System?	____	____	_____
C. Culverts?	____	____	_____
D. Dry Detention?	____	____	_____
E. Energy Dissipation?	____	____	_____
F. Enhanced Swales?	____	____	_____
G. Extended Dry Detention?	____	____	_____
H. Filter Strips?	____	____	_____
I. Grass Channels?	____	____	_____
J. Gravity (Oil-Grit) Separator?	____	____	_____
K. Infiltration Trenches?	____	____	_____
L. Modular Porous Paver Systems?	____	____	_____
M. Multi-Purpose Detention Areas?	____	____	_____
N. Open Channels?	____	____	_____
O. Organic Filters?	____	____	_____
P. Porous Concrete?	____	____	_____
Q. Proprietary Systems?	____	____	_____
R. Sand Filters (Surface/Perimeter)?	____	____	_____
S. Storm Water Ponds?	____	____	_____
T. Storm Water Wetlands?	____	____	_____
U. Street Gutters/Inlets/Pipe Systems?	____	____	_____
V. Submerged Gravel Wetland?	____	____	_____
W. Underground Detention?	____	____	_____
X. Underground Sand Filters?	____	____	_____
Y. Other controls?	____	____	_____

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Checklist for Conceptual iSWM Site Plan Preparation and Review

	Included? Yes	No	Comments
Mapping and plans which illustrate at a minimum: (recommended scale of 1" = 50' or greater)			
1. Project Description			
A. Name, legal address, & telephone number of applicant	____	____	_____
B. Name, legal address & telephone number of preparer	____	____	_____
C. Common address and legal description of site	____	____	_____
D. Vicinity map	____	____	_____
E. Proposed land use with Standard Industrial Code No.	____	____	_____

	Yes	No	Comments
2. Existing Conditions			
A. Copy of applicable digital orthophoto showing proposed project boundaries	____	____	_____
B. Best available existing topography (no greater than 2-foot contours recommended)	____	____	_____
C. Total Site Area and Total Impervious Area (acres)	____	____	_____
D. Benchmarks used for site control if available	____	____	_____
E. Perennial and intermittent streams	____	____	_____
F. Predominant soils from USDA soil surveys and/or on site soil borings	____	____	_____
G. Boundaries of existing predominant vegetation	____	____	_____
H. Location and boundaries of natural feature protection and conservation areas such as wetlands, lakes, ponds, and other setbacks (e.g., stream buffers, drinking water well setbacks, septic setbacks, etc.)	____	____	_____
I. Location of existing roads, buildings, parking lots and other impervious areas	____	____	_____
J. Location of existing utilities (e.g., water, sewer, gas, electric) and easements	____	____	_____
K. Location of existing conveyance systems such as storm drains, inlets, catch basins, channels, swales, and areas of overland flow	____	____	_____
L. Flow paths	____	____	_____
M. Location of floodplain/floodway limits and relationship of site to upstream/downstream properties and drainages	____	____	_____
N. Location and dimensions of existing channels, bridges or culvert crossings	____	____	_____

Checklist for Conceptual iSWM Site Plan Preparation and Review (Continued)

	Yes	No	Comments
3. Conceptual Site Layout			
A. Complete the iSWM Conceptual Plan Worksheet	____	____	_____
B. Hydrologic analysis to determine conceptual runoff rates, volumes and velocities to support selection of Storm Water Controls	____	____	_____
C. Conceptual site design identifying *integrated* site design practices used	____	____	_____
D. Identification and calculation of storm water site design credits	____	____	_____
E. Conceptual estimates of *integrated* Design Approach requirements	____	____	_____
F. Conceptual selection, location and size of proposed structural storm water controls	____	____	_____
G. Conceptual limits of proposed clearing and grading	____	____	_____

Checklist for Preliminary iSWM Site Plan Preparation and Review

	Included? Yes	No	Comments
Mapping and plans which illustrate at a minimum: (recommended scale of 1" = 50' or greater)			
1. Existing Conditions Hydrologic Analysis			
A. Existing and proposed topography (no greater than 2-foot contours recommended)	____	____	_____
B. Total Site Area and Total Impervious Area (acres)	____	____	_____
C. Perennial and intermittent streams	____	____	_____
D. Predominant soils from USDA soil surveys or soil borings	____	____	_____
E. Boundaries of existing predominant vegetation and proposed limits of clearing and grading	____	____	_____
F. Location and boundaries of natural feature protection and conservation areas such as wetlands, lakes, ponds, and other setbacks (e.g., stream buffers, drinking water well setbacks, septic setbacks, etc.)	____	____	_____
G. Location of existing and proposed roads, buildings, parking lots and other impervious areas	____	____	_____
H. Location of existing and proposed utilities (e.g., water, sewer, gas, electric) and easements	____	____	_____
I. Preliminary selection and location of storm water controls	____	____	_____
J. Location of existing and proposed conveyance systems such as storm drains, inlets, catch basins, channels, swales, and areas of overland flow	____	____	_____
K. Flow paths	____	____	_____
L. Location of floodplain/floodway limits and relationship of site to upstream/downstream properties and drainages	____	____	_____
M. Preliminary location and dimensions of proposed channel modifications, such as bridge or culvert crossings	____	____	_____
N. Existing conditions hydrologic analysis for runoff rates, volumes and velocities showing methodologies used and supporting calculations	____	____	_____

Checklist for Preliminary iSWM Site Plan Preparation and Review (Continued)

	Included? Yes	No	Comments
2. Project Description and Design Considerations (updated information from Conceptual Plan)			
A. Name, legal address and telephone number of applicant	____	____	_____
B. Name, legal address and telephone number of preparer	____	____	_____
C. Common address and legal description of site	____	____	_____
D. Vicinity map	____	____	_____
E. Discussion of *integrated* Site Design Practices	____	____	_____
F. Discussion of Credits for *integrated* Site Design	____	____	_____
G. Discussion of storm water controls	____	____	_____
H. Discussion of groundwater recharge considerations	____	____	_____
I. Discussion of hotspot land uses and runoff treatment	____	____	_____

	Yes	No	Comments
3. Post-Development Hydrologic Analysis			
A. Proposed (post-development) conditions hydrologic analysis for runoff rates, volumes, and velocities showing the methodologies used and supporting calculations	____	____	_____
B. Preliminary estimates of *integrated* Design Approach requirements	____	____	_____
C. Preliminary identification and calculation of credits for *integrated* site designs	____	____	_____
D. Location and boundary of proposed natural feature protection areas	____	____	_____

	Yes	No	Comments
4. Downstream Assessments			
A. Preliminary analysis of potential downstream impact/effects of project, where necessary	____	____	_____

	Yes	No	Comments
5. Storm Water Management System Design			
A. Hydrologic and hydraulic analysis of the storm water management system for all applicable design storms	____	____	_____
B. Preliminary sizing calculations for storm water controls including contributing drainage area, storage, and outlet configuration	____	____	_____
C. Narrative describing the selected storm water controls	____	____	_____

Checklist for Final iSWM Site Plan Preparation and Review

	Included? Yes	No	Comments
1. Existing Conditions Hydrologic Analysis			
A. Updated checklist from **Preliminary iSWM Site Plan**.....	____	____	_____
2. Project Description and Design Considerations			
A. Updated checklist from **Preliminary iSWM Site Plan**.....	____	____	_____
3. Post-Development Hydrologic Analysis			
A. Updated checklist from **Preliminary iSWM Site Plan**.....	____	____	_____
B. Final sizing calculations for storm water controls including contributing drainage area, storage, and outlet configuration.....	____	____	_____
C. Stage-discharge or outlet rating curves and inflow and outflow hydrographs for storage facilities.....	____	____	_____
D. Final analysis of potential downstream impact/effects of project, where necessary	____	____	_____
E. Dam safety and breach analysis, where necessary	____	____	_____
4. Downstream Assessments			
A. Update checklist from **Preliminary iSWM Site Plan**.....	____	____	_____
	Yes	**No**	**Comments**
5. Storm Water Management System Design			
A. Update checklist from **Preliminary iSWM Site Plan**.....	____	____	_____
B. Existing and proposed structural elevations (e.g., invert of pipes, manholes, etc.).....	____	____	_____
C. Design water surface elevations	____	____	_____
D. Structural details and specifications of structural control designs, outlet structures, embankments, spillways, grade control structures, conveyance channels, etc.....	____	____	_____
E. Professional Engineer seal, signature and date	____	____	_____
6. Construction Storm Water Pollution Prevention Plan			
A. Required elements specified in iSWM Design Manual for Construction and/or local ordinances.....	____	____	_____
B. Sequence/phasing of construction and temporary stabilization measures.....	____	____	_____
C. Temporary structures that will be converted into permanent storm water controls	____	____	_____

Checklist for Final iSWM Site Plan (Continued)

	Included? Yes	No	Comments
7. Landscaping Plan			
A. Arrangement of planted areas, natural areas, and other landscaped features	____	____	_____
B. Information required to construct landscaping elements	____	____	_____
C. Descriptions and standards for the methods, materials and vegetation that are to be used	____	____	_____

	Yes	No	Comments
8. Operations and Maintenance Plan			
A. Name, legal address and phone number of responsible parties for maintenance activities	____	____	_____
B. Description and schedule of maintenance tasks	____	____	_____
C. Description of applicable easements	____	____	_____
D. Description of funding source	____	____	_____
E. Access and safety issues	____	____	_____
F. Procedures for testing and disposal of sediments, if required	____	____	_____
G. Expected service life of structures and estimated cost to replace	____	____	_____
H. Executed Maintenance Agreement(s), as required	____	____	_____

	Yes	No	Comments
9. Evidence of Acquisition of Applicable Federal, State, and Local Permits			
A. USACE Regulatory Program permits	____	____	_____
B. 401 water quality certification	____	____	_____
C. TPDES Construction permit	____	____	_____
D. Other_____	____	____	_____
E. Other_____	____	____	_____

	Yes	No	Comments
10. Waiver requests			
A. Evidence of acquisition of all necessary legal agreements (e.g., easements, covenants, land trusts, etc.)	____	____	_____

Operation and Maintenance Inspection Report for Storm Water Management Ponds

(Adapted from Watershed Management Institute, Inc.)

Inspector Name _____ Project Location
Inspection Date _____ _____
Storm Water Pond _____
Normal Pool _____ _____
Normally Dry _____ Watershed _____
Embankment Type: Earthen ☐ Concrete ☐

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
Pond Components				
1. Embankment and/or Emergency spillway				
a. Adequate vegetation and ground cover			A	
b. Embankment erosion			A	
c. Animal burrows			A	
d. Unauthorized plantings			A	
e. Cracking, bulging, or sliding of dam				
i. Upstream face			A	
ii. Downstream face			A	
iii. At or beyond toe Upstream			A	
Downstream			A	
iv. Emergency spillway			A	
f. Pond, toe & chimney drains clear and functioning			A	
g. Leaks on downstream face			A	
h. Abutment protection or riprap failures			A	
i. Visual settlement or horizontal misalignment of top of dam			A	
j. Emergency spillway clear of debris			A	
k. Other (specify)			A	
2. Riser and principal spillway				
Type: Reinforced concrete _ Corrugated pipe _____ Masonry _____				
a. Low flow orifice obstructed			A	
b. Low flow trash rack			A	
i. Debris removal necessary				
ii. Corrosion control			A	

Inspection Items			Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
	c.	Weir trash rack i. Debris removal necessary			**A**	
		ii. Corrosion control			**A**	
	d.	Excessive sediment accumulation inside riser			**A**	
	e.	Concrete/Masonry condition Riser and barrels i. Cracks or displacement			**A**	
		ii. Minor spalling (<1")			**A**	
		iii. Major spalling (rebars exposed)			**A**	
		iv. Joint failures			**A**	
		v. Water tightness			**A**	
	f.	Metal pipe condition			**A**	
	g.	Control valve i. Operational/exercised			**A**	
		ii. Chained and locked			**A**	
	h.	Pond drain valve			**A**	
		i. Operational/exercised			**A**	
		ii. Chained and locked			**A**	
	i.	Outfall channels flowing			**A**	
	j.	Other (specify)			**A**	
3.	Permanent pool (wet ponds)					
	a.	Undesirable vegetative growth			**M**	
	b.	Floating or floatable debris removal required			**M**	
	c.	Visible pollution			**M**	
	d.	High Water Marks			**M**	
	e.	Shoreline problems			**M**	
	f.	Sediment build-up			**M**	
	g.	Other (specify)			**M**	
4.	Sediment forebays					
	a.	Sedimentation Noted			**M**	
	b.	Sediment removal when depth > 50% design depth			**M**	
5.	Dry pond areas					
	a.	Vegetation adequate			**M**	
	b.	Undesirable vegetative growth			**M**	
	c.	Undesirable woody vegetation			**M**	
	d.	Low flow channels clear of obstructions			**M**	
	e.	Standing water or wet spots			**M**	
	f.	Sediment and/or trash accumulation			**M**	
	g.	Other (specify)			**M**	
6.	Condition of outfalls into pond					
	a.	Riprap failures			**A,S**	

Inspection Items		Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
b.	Slope erosion			**A,S**	
c.	Storm drain pipes			**A,S**	
d.	Endwalls/headwalls			**A,S**	
e.	Other (specify)			**A,S**	
7. Other					
a.	Encroachments on ponds or easement area			**M**	
b.	Complaints from residents (describe on back)			**M**	
c.	Aesthetics i. Grass height			**M**	
	ii. Graffiti removal necessary			**M**	
	iii. Other (specify)			**M**	
d.	Any public hazards (specify)			**M**	
e.	Maintenance access			**M**	
8. Constructed wetland areas					
a.	Vegetation healthy and growing			**A**	
b.	Evidence of invasive species			**A**	
c.	Excessive sedimentation in wetland area			**A**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Summary

1. Inspectors Remarks: _____
_____
_____
_____
_____
_____
_____

2. Overall condition of Facility (Check one)
   _____ Acceptable
   _____ Unacceptable

3. Dates any maintenance must be completed by:
_____
_____
_____

_____
Inspectors Signature

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Operation and Maintenance Inspection Report for Filtration Facility

(Adapted from Watershed Management Institute, Inc.)

Inspector Name _____ Project Location
Inspection Date _____ _____
Watershed _____ _____
As-built Plans available? _____ _____

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
1. Debris removal				
Adjacent area clear of debris			**M**	
Inlets and outlets clear of debris			**M**	
Filtration facility free of debris			**M**	
Illegal dumping			**M**	
Vandalism			**M**	
2. Vegetation				
Adjacent area stabilized			**M**	
Grass mowed			**M**	
Any evidence of erosion			**M**	
3. Oil and grease				
Any evidence of filter clogging			**M**	
4. Water retention where required				
Water holding chambers at normal pool			**M**	
No evidence of leakage			**M**	
5. Sediment deposition				
Filtration chamber clean of sediments			**A**	
Water chambers not more than ½ full of sediments			**A**	
6. Structural components				
Any evidence of structural deterioration			**A**	
Grates in good condition			**A**	
Any evidence of spalling or cracking of structural parts			**A**	
7. Outlets/overflow spillway				
Good condition (no need for repair)			**A**	
Any evidence of erosion			**A**	
8. Overall function of facility				
Any evidence of flow bypassing facility			**A**	
Any noticeable odors outside of facility			**A**	

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
9. Pump (Where applicable)				
Catalog cuts and wiring diagram for pump available			**A**	
Waterproof conduits for wiring appear to be intact			**A**	
Panel box is well marked			**A**	
Any evidence of pump failure (excess water in pump well, etc.)			**A**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Necessary Action:

If any of the items above where answered Yes for "Maintenance Needed", a time frame needs to be established for repair or correction.

No action necessary. Continue routine inspections.
Correct noted facility deficiencies by (date) _____

Facility repairs were previously indicated and completed. Site reinspection is necessary to verify corrections or improvements.

Site reinspection completed on (date) _____

Site reinspection was satisfactory.
Next routine inspection is scheduled for approximately (date):_____

_____
Inspectors Signature

Operation and Maintenance Inspection Report for Infiltration Trenches

(Adapted from Watershed Management Institute, Inc.)

Inspector Name _____ Project Location
Inspection Date _____ _____
Watershed _____ _____
As-built Plans available? _____ _____

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
1. Debris removal				
Trench surface clear of debris			**M**	
Inlets clear of debris			**M**	
Inflow pipes clear of debris			**M**	
Overflow spillway clear of debris			**M**	
2. Sediment traps, forebays, or pretreatment swales				
Obviously trapping sediment			**A**	
Greater than 50% of original storage volume remaining			**A**	
3. Vegetation				
Mowing done when necessary			**M**	
Fertilized per specification			**M**	
Any evidence of erosion			**M**	
Contributing drainage area stabilized			**M**	
4. Dewatering				
Trench dewaters between storms			**M**	
5. Sediment removal of trench				
Any evidence of sedimentation in trench			**A**	
Does sediment accumulation currently require removal			**A**	
6. Inlets				
Good condition			**A**	
Any evidence of erosion			**A**	
7. Outlets/overflow spillway				
Good condition (no need for repair)			**A**	
Any evidence of erosion			**A**	
8. Aggregate repairs				
Surface of aggregate clean			**A**	
Top layer of stone in need of replacement			**A**	
Trench in need of rehabilitation			**A**	

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
9. Vegetated surface				
Evidence of erosion present			**M**	
Perforated inlet functioning adequately			**M**	
Does water stand on vegetated surface			**M**	
Does good vegetative cover exist			**M**	
10. Overall function of facility				
Any evidence of flow bypassing facility			**S**	
Water level in monitoring well			**S**	
Damage to cap or well			**S**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Necessary Action:

If any of the items above where answered Yes for "Maintenance Needed", a time frame needs to be established for repair or correction.

No action necessary. Continue routine inspections.
Correct noted facility deficiencies by (date) _____

Facility repairs were previously indicated and completed. Site reinspection is necessary to verify corrections or improvements.

Site reinspection completed on (date) _____

Site reinspection was satisfactory.
Next routine inspection is scheduled for approximately (date):_____

_____
Inspectors Signature

Operation and Maintenance Inspection Report for Enhanced Swales / Grass Channels / Filter Strips

(Adapted from Watershed Management Institute, Inc.)

Inspector Name _____ Project Location
Inspection Date _____ _____
Watershed _____ _____
As-built Plans available? _____ _____

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
1. Debris removal Facility and adjacent area clear of debris			**M**	
Inlets and outlets clear of debris			**M**	
Any dumping of yard wastes into facility			**M**	
Has litter (branches, etc.) been removed			**M**	
2. Vegetation Adjacent area stabilized			**M**	
Grass mowed			**M**	
Plant height not less than design water depth			**M**	
Fertilized per specifications			**M**	
Any evidence of erosion			**M**	
Is plant composition according to approved plans			**M**	
Any unauthorized or inappropriate plantings			**M**	
Any dead or diseased plants			**M**	
Any evidence of plant stress from inadequate watering			**M**	
Any evidence of deficient stakes or wires			**M**	
3. Oil and grease Any evidence of filter clogging			**M**	
4. Dewatering Facility dewaters between storms			**M**	
5. Check dams/energy dissipators/sumps Any evidence of sedimentation buildup			**A,S**	
Are sumps greater than 50% full of sediment			**A,S**	
Any evidence of erosion at downstream toe of drop structures			**A,S**	

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
6. Sediment deposition Swale clean of sediments			**A**	
Sediments should not be > than 20% of swale design depth			**A**	
7. Outlets/overflow spillway Good condition (no need for repair)			**A,S**	
Any evidence of erosion			**A,S**	
Any evidence of blockages			**A,S**	
8. Integrity of facility Has facility been blocked or filled inappropriately			**A**	
9. Bioretention Planting Soil Any evidence of planting soil erosion			**A**	
10. Organic Layer Mulch covers entire area (NO voids) and to specified thickness			**A**	
Mulch is in good condition			**A**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Necessary Action:

If any of the items above where answered Yes for “Maintenance Needed”, a time frame needs to be established for repair or correction.

No action necessary. Continue routine inspections.
Correct noted facility deficiencies by (date) _____

Facility repairs were previously indicated and completed. Site reinspection is necessary to verify corrections or improvements.

Site reinspection completed on (date) _____

Site reinspection was satisfactory.
Next routine inspection is scheduled for approximately (date):_____

_____
Inspectors Signature

APPENDIX

F

LANDSCAPING AND AESTHETICS GUIDANCE

Introduction

Landscaping is a critical element in the design of storm water facilities for water quantity and quality management, serving both functional and aesthetic purposes. Plants and vegetation perform a number of functions in storm water controls and conveyance facilities, including:

- Slowing and retarding flow by increasing hydraulic roughness
- Preventing the erosion of bare soil
- Enhancing infiltration of runoff into the soil
- Providing pollutant removal through vegetative uptake
- Preventing access to deep open water areas
- Contributing to wildlife and fish habitat
- Improving the overall appearance of storm water facilities

The purpose of this Appendix is to provide guidance on landscaping and plant selection for storm water facilities and structural controls, as well as provide an overview on developing aesthetically-pleasing storm water facilities. This appendix is divided into the following sections:

- Subsection F.1 covers general landscaping guidance that should be considered when landscaping any storm water facility.
- Subsection F.2 discusses the physical site factors and considerations involved in selecting plant material for storm water facility landscaping.
- Subsection F.3 includes key factors to consider in selecting plant material for storm water landscaping are reviewed, including hardiness, physiographic regions, inundation tolerance, and other factors.
- Subsection F.4 outlines more specific guidance on landscaping criteria and plant selection for individual structural storm water control designs, including:
    - Storm Water Ponds and Wetlands
    - Bioretention Areas
    - Infiltration Trench and Surface Sand Filter Facilities
    - Enhanced Swales and Grass Channels
    - Filter Strips and Stream Buffers
    - Green Roofs
- Subsection F.5 contains a detailed plant list of trees and shrubs that may be used when preparing a vegetation and landscaping planting plan for a storm water facility.
- Subsection F.6 provides examples of aesthetics and good landscaping in structural control design.

For information on native and adapted plants and trees to create landscapes that need less water, pesticides, and fertilizer to thrive in the North Central Texas climate, visit www.txsmartscape.com. Excessive fertilizer in runoff can lead to “overgrowth” of submerged plants.

F.1 General Landscaping Guidance

Below are general guidelines that should be followed in the landscaping of any storm water control or conveyance facility.

DO NOT:

- Plant trees, scrubs or any type of woody vegetation on an embankment
- Plant trees and shrubs within 15 feet of the toe of slope of a dam.
- Plant trees or shrubs known to have long tap roots within the vicinity of the earthen dam or embankment, or subsurface drainage facilities.
- Plant trees and shrubs within 25 feet of a principal spillway structure (e.g., riser)
- Plant trees and shrubs within 25 feet of perforated pipes.
- Block maintenance access to structures with trees or shrubs.
- Plant trees and shrubs within 25 feet of a structural concrete dam.

DO:

- Take into account site characteristics and plant selection guidelines (see subsections F.2 and F.3, respectively) when selecting plants for storm water facilities.
- Consider how plant characteristics will affect the landscape and the performance of a structural storm water control or conveyance.
- Carefully consider the long-term vegetation management strategy for the structural control, keeping in mind the maintenance legacy for the future owners.
- Preserve existing natural vegetation when possible.
- Avoid the overuse of any plant materials.
- Have soils tested to determine if there is a need for amendments.
- Select plants that can thrive in on-site soils with no additional amendments or a minimum of amendments.
- Consider water availability, particularly for wetland and water-intensive plantings.
- Decrease the areas where turf is used. Use low maintenance ground cover to absorb run-off.
- Plant stream and edge of water buffers with trees, shrubs, ornamental grasses, and herbaceous materials where possible, to stabilize banks and provide shade.
- Provide slope stabilization methods for slopes steeper than 2:1, such as planted erosion control mats. Also, use seed mixes with quick germination rates in this area. Augment temporary seeding measures with container crowns or root mats of more permanent plant material.
- Utilize erosion control mats and fabrics to protect inverts of channels that are subject to frequent wash outs.
- Stabilize all water overflows with plant material that can withstand strong current flows. Root material should be fibrous and substantial but lacking a tap root.
- Sod area channels that are not stabilized using erosion control mats.
- Divert flows temporarily from seeded areas until stabilized.
- Check water tolerances of existing plant materials prior to inundation of area.
- Stabilize aquatic and safety benches with emergent wetland plants and wet seed mixes.
- Provide a 15-foot clearance from a non-clogging, low flow orifice.

- Limit herbaceous embankment plantings to 10 inches in height, to allow visibility for the inspector who is looking for burrowing rodents that may compromise the integrity of the embankment.
- Shade inflow and outflow channels, as well as the southern exposures of pond, to reduce thermal warming
- Avoid plantings that will require routine or intensive chemical applications (i.e. turf area).
- Maintain and frame desirable views. Be careful not to block views at entrances, exits, or difficult road curves. Screen or buffer unattractive views into the site.
- Use plants to prohibit pedestrian access to pools or slopes that may be unsafe.
- Keep maintenance area open to allow future access for pond maintenance.
- Provide a planting surface that can withstand the compaction of vehicles using maintenance access roads.
- Make sure the facility maintenance agreement includes a maintenance requirement of designated plant material.
- Provide signage for:
  - Storm water management facilities to help educate the public
  - Wildflower areas to designate limits of mowing
  - Preserving existing natural vegetation

F.2 Site Considerations

A development site's characteristics often will help to determine which plant materials and planting methods the site designer should select and will help improve plant establishment. Primary site considerations include:

(1) Soil Characteristics
(2) Drainage
(3) Slope
(4) Orientation

Soil Characteristics

Plant establishment and growth can be limited by a number of different soil characteristics including:

- Soil texture
- PH -- whether acid, neutral, or alkali
- Nutrient levels -- nitrogen, phosphorus, potassium
- Minerals -- such as chelated iron, lime
- Salinity
- Toxicity

Soils are made up of four basic ingredients: mineral elements, pore space, organic matter and other items consisting mainly of living organisms including fungi, bacteria, and nematodes. One classification of soils is based upon the mineral part of soil and consists of four sizes of particles. Clay particles are the smallest, followed by silt, sand, and gravel. The USDA has devised another system of classifying soil particles. In this system soil is divided into seven categories: clay, silt, and five sizes of sand.

Soil texture is determined by the percentage of sand, silt, and clay in the soil. The structure of a soil is influenced by soil texture and also by the aggregation of small soil particles into larger particles. The amount of aggregation in a soil is strongly influenced by the amount of organic matter present.

Soil samples should be analyzed by experienced and qualified individuals who can explain the results and provide information on any soil amendments that are required. Soil fertility can often be corrected by applying fertilizer or by increasing the level of organic matter in the soil. Soil pH can be corrected with applications of lime. Where poor soils can't be amended, seed mixes and plant material must be selected to establish ground cover as quickly as possible.

Areas that have recently been involved in construction can become compacted so that plant roots cannot penetrate the soil. Seeds lying on the surface of compacted soils can be washed away or be eaten by birds. Soils should be loosened to a minimum depth of two inches, preferably to a four-inch depth. Hard soils may require discing to a deeper depth. Loosening soils will improve seed contact with the soil, provide greater germination rates, and allow the roots to penetrate into the soil. If the area is to be sodded, discing will allow the roots to penetrate into the soil.

Whenever possible, topsoil should be spread to a depth of four inches (two inch minimum) over the entire area to be planted. This provides organic matter and important nutrients for the plant material. This also allows the stabilizing materials to become established faster, while the roots are able to penetrate deeper and stabilize the soil, making it less likely that the plants will wash out during a heavy storm. If topsoil has been stockpiled in deep mounds for a long period of time, it is desirable to test the soil for pH as well as microbial activity. If the microbial activity has been destroyed, it may be necessary to inoculate the soil after application.

Drainage

Soil moisture and drainage have a direct bearing on the plant species and communities that can be supported on a site. Factors such as soil texture, topography, groundwater levels and climatic patterns all influence soil drainage and the amount of water in the soil. Identifying the topography and drainage of the site will help determine potential moisture gradients. The following categories can be used to describe the drainage properties of soils on a site:

Flooded - Areas where standing water is present most of the growing season.

Wet - Areas where standing water is present most of the growing season, except during times of drought. Wet areas are found at the edges of ponds, rivers, streams, ditches, and low spots. Wet conditions exist on poorly drained soils, often with a high clay content.

Moist - Areas where the soil is damp. Occasionally, the soil is saturated and drains slowly. These areas usually are at slightly higher elevations than wet sites. Moist conditions may exist in sheltered areas protected from sun and wind.

Well-drained - Areas where rain water drains readily and puddles do not last long. Moisture is available to plants most of the growing season. Soils usually are medium textures with enough sand and silt particles to allow water to drain through the soil.

Dry - Areas where water drains rapidly through the soil. Soils are usually coarse, sandy, rocky or shallow. Slopes are often steep and exposed to sun and wind. Water runs off quickly and does not remain in the soil.

Slope

The degree of slope can also limit its suitability for certain types of plants. Plant establishment and growth requires stable substrates for anchoring root systems and preserving propagules such as seeds and plant fragments, and slope is a primary factor in determining substrate stability. Establishing plants directly on or below eroding slopes is not possible for most species. In such instances, plant species capable of rapid spread and anchoring soils should be selected or bioengineering techniques should be used to aid the establishment of a plant cover.

In addition, soils on steep slopes generally drain more rapidly than those on gradual slopes. This means that the soils may remain saturated longer on gradual slopes. If soils on gradual slopes are classified as poorly drained, care should be taken that plant species are selected that are tolerant of saturation.

Site topography also affects maintenance of plant species diversity. Small irregularities in the ground surface (e.g., depressions, etc.) are common in natural systems. More species are found in areas with many micro-topographic features than in areas without such features. Raised sites are particularly important in wetlands because they allow plants that would otherwise die while flooded to escape inundation.

In wetland plant establishment, ground surface slope interacts with the site hydrology to determine water depths for specific areas within the site. Depth and duration of inundation are principal factors in the zonation of wetland plant species. A given change in water levels will expose a relatively small area on a steep slope in comparison with a much larger area exposed on a gradual or flat slope. Narrow planting zones will be delineated on steep slopes for species tolerant of specific hydrologic conditions, whereas gradual slopes enable the use of wider planting zones.

Orientation

Slope exposure should be considered for its effect on plants. A southern-facing slope receives more sun and is warmer and drier, while the opposite is true of a northern slope. Eastern- and western-facing slopes are intermediate, receiving morning and afternoon sun, respectively. Western-facing slopes tending to receive more wind.

F.3 Plant Selection for Storm Water Facilities

F.3.1 Hardiness Zones

Hardiness zones are based on historical annual minimum temperatures recorded in an area. A site's location in relation to plant hardiness zones is important to consider first because plants differ in their ability to withstand very cold winters. This does not imply that plants are not affected by summer temperatures. Given that Texas summers can be very hot, heat tolerance is also a characteristic that should be considered in plant selection.

It is best to recommend plants known to thrive in specific hardiness zones. The plant list included at the end of this appendix identifies the hardiness zones for each species listed as a general planting guide. It should be noted, however, that certain site factors can create microclimates or environmental conditions which permit the growth of plants not listed as hardy for that zone. By investigating numerous references and based on personal experience, a designer should be able to confidently recommend plants that will survive in microclimates.

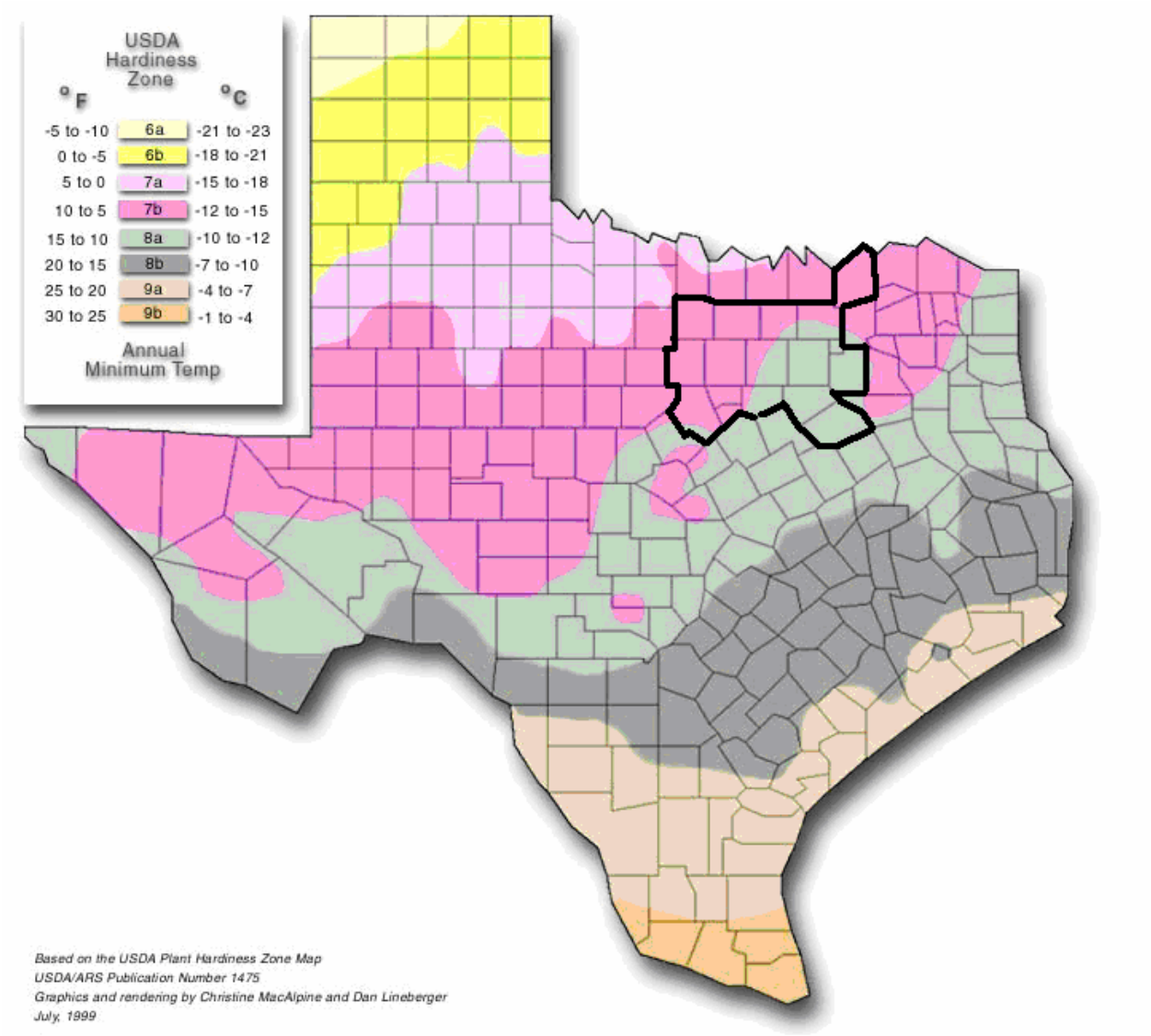
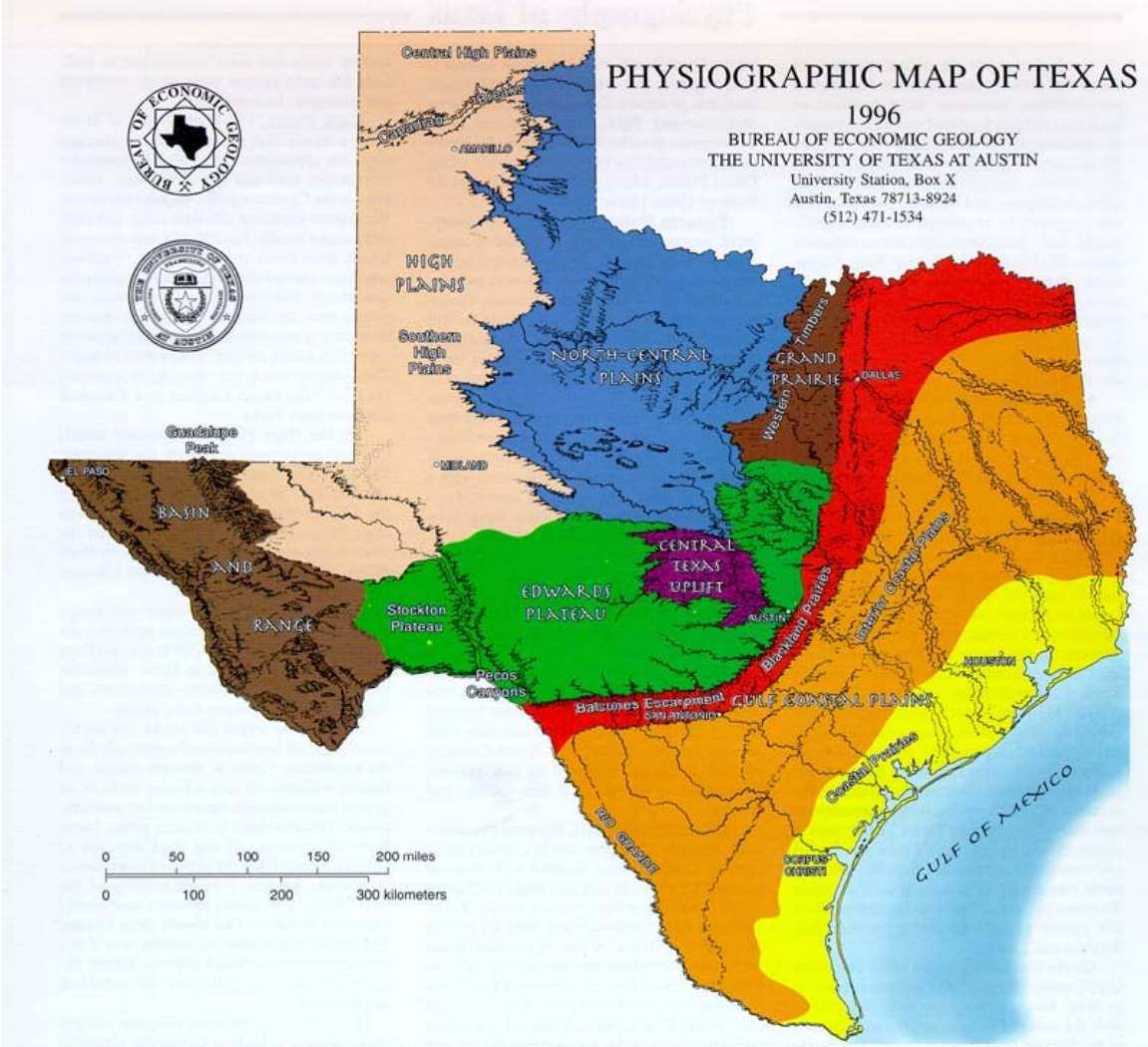


Figure F-1 USDA Plant Hardiness Zones in Texas

F.3.2 Physiographic Provinces

There are three physiographic provinces in Texas that describe distinct geographic regions in the state with similar physical and environmental conditions (Figure F-2). These physiographic provinces include, from northwest to southeast, High Plains, Edwards Plateau, and Gulf Coastal Plains (subdivided into multiple subregions). Each physiographic region is defined by unique geological strata, soil type, drainage patterns, moisture content, temperature and degree of slope which often dictate the predominant vegetation. Because the predominant vegetation has evolved to live in these specific conditions, a successful storm water management facility planting design can be achieved through mimicking these natural associations. The three physiographic regions are described below with associated vegetation listed as general planting guidance.



PROVINCE	MAX. ELEV. (ft)	MIN. ELEV. (ft)	TOPOGRAPHY	GEOLOGIC STRUCTURE	BEDROCK TYPES
Gulf Coastal Plains					
Coastal Prairies	300	0	Nearly flat prairie, <1 ft/mi to Gulf	Nearly flat strata	Deltaic sands and muds
Interior Coastal Plains	800	300	Parallel ridges (questas) and valleys	Beds tilted toward Gulf	Unconsolidated sands and muds
Blackland Prairies	1000	450	Low rolling terrain	Beds tilted south and east	Chalks and marls
Grand Prairie	1250	450	Low stairstep hills west; plains east	Strata dip east	Calcareous east; sandy west
Edwards Plateau					
Principal	3000	450	Flat upper surface with box canyons	Beds dip south; normal faulted	Limestones and dolomites
Pecos Canyons	2000	1200	Steep-walled canyons		Limestones and dolomites
Stockton Plateau	4200	1700	Mesa-formed terrain; highs to west	Unfaulted, near-horizontal beds	Carbonates and alluvial sediments
Central Texas Uplift	2000	800	Knobby plain; surrounded by questas	Centripetal dips, strongly faulted	Granites; metamorphics; sediments
North-Central Plains	3000	900	Low north-south ridges (questas)	West dip; minor faults	Limestones; sandstones; shales
High Plains					
Central	4750	2900	Flat prairies slope east and south	Slight dips east and south	Eolian silts and fine sands
Canadian Breaks	3800	2350	Highly dissected; local solution valleys		
Southern	3800	2200	Flat; many playas; local dune fields		
Basin and Range	8750	1700	North-south mountains and basins	Some complex folding and faulting	Igneous; metamorphics; sediments

Figure F-2 Physiographic map of Texas

Gulf Coastal Plains. The Gulf Coastal Plains include three subprovinces named the Coastal Prairies, the Interior Coastal Plains, and the Blackland Prairies. The Coastal Prairies begin at the Gulf of Mexico shoreline. Young deltaic sands, silts, and clays erode to nearly flat grasslands that form almost imperceptible slopes to the southeast. Trees are uncommon except locally along streams and in Oak mottes, growing on coarser underlying sediments of ancient streams. Minor steeper slopes, from 1 foot to as much as 9 feet high, result from subsidence of deltaic sediments along faults. Between Corpus Christi

and Brownsville, broad sand sheets pocked by low dunes and blowouts forming ponds dominate the landscape.

The Interior Coastal Plains comprise alternating belts of resistant uncemented sands among weaker shales that erode into long, sandy ridges. At least two major down-to-the coast fault systems trend nearly parallel to the coastline. Clusters of faults also concentrate over salt domes in East Texas. That region is characterized by pine and hardwood forests and numerous permanent streams. West and south, tree density continuously declines, pines disappear in Central Texas, and chaparral brush and sparse grasses dominate between San Antonio and Laredo.

On the Blackland Prairies of the innermost Gulf Coastal Plains, chalks and marls weather to deep, black, fertile clay soils, in contrast with the thin red and tan sandy and clay soils of the Interior Gulf Coastal Plains. The blacklands have a gentle undulating surface, cleared of most natural vegetation and cultivated for crops.

From sea level at the Gulf of Mexico, the elevation of the Gulf Coastal Plains increases northward and westward. In the Austin San Antonio area, the average elevation is about 800 feet. South of Del Rio, the western end of the Gulf Coastal Plains has an elevation of about 1,000 feet.

Grand Prairie. The eastern Grand Prairie developed on limestones; weathering and erosion have left thin rocky soils. North and west of Fort Worth, the plateau like surface is well exposed, and numerous streams dissect land that is mostly flat or that gently slopes southeastward. There, silver bluestem-Texas wintergrass grassland is the flora. Primarily sandstones underlie the western margin of the Grand Prairie, where post oak woods form the Western Cross Timbers.

Edwards Plateau. The Balcones Escarpment, superposed on a curved band of major normal faults, bounds the eastern and southern Edwards Plateau. Its principal area includes the Hill Country and a broad plateau. Stream erosion of the fault escarpment sculpts the Hill Country from Waco to Del Rio. The Edwards Plateau is capped by hard Cretaceous limestones. Local streams entrench the plateau as much as 1,800 feet in 15 miles. The upper drainages of streams are waterless draws that open into box canyons where springs provide permanently flowing water. Sinkholes commonly dot the limestone terrain and connect with a network of caverns. Alternating hard and soft marly limestones form a stair-step topography in the central interior of the province.

The Edwards Plateau includes the Stockton Plateau, mesa like land that is the highest part of this subdivision. With westward decreasing rainfall, the vegetation grades from mesquite juniper brush westward into creosote bush tarbush shrubs.

The Pecos River erodes a canyon as deep as 1,000 feet between the Edwards and Stockton Plateaus. Its side streams become draws forming narrow blind canyons with nearly vertical walls. The Pecos Canyons include the major river and its side streams. Vegetation is sparse, even near springs and streams.

Central Texas Uplift. The most characteristic feature of this province is a central basin having a rolling floor studded with rounded granite hills 400 to 600 feet high. Enchanted Rock State Park is typical of this terrain. Rocks forming both basin floor and hills are among the oldest in Texas. A rim of resistant lower Paleozoic formations surrounds the basin. Beyond the Paleozoic rim is a second ridge formed of limestones like those of the Edwards Plateau. Central live oak mesquite parks are surrounded by live oak ash juniper parks.

North-Central Plains. An erosional surface that developed on upper Paleozoic formations forms the North-Central Plains. Where shale bedrock prevails, meandering rivers traverse stretches of local prairie. In areas of harder bedrock, hills and rolling plains dominate. Local areas of hard sandstones and limestones cap steep slopes severely dissected near rivers. Lengthy dip slopes of strongly fractured limestones display extensive rectangular patterns. Western rocks and soils are oxidized red or gray where gypsum dominates, whereas eastern rocks and soils weather tan to buff. Live oak ash juniper parks grade westward into mesquite lotebush brush.

High Plains. The High Plains of Texas form a nearly flat plateau with an average elevation approximating 3,000 feet. Extensive stream-laid sand and gravel deposits, which contain the Ogallala aquifer, underlie the plains. Windblown sands and silts form thick, rich soils and caliche locally. Havard shin oak mesquite brush dominates the silty soils, whereas sandsage Havard shin oak brush occupies the sand sheets. Numerous playa lakes scatter randomly over the treeless plains. The eastern boundary is a westward-retreating escarpment capped by a hard caliche. Headwaters of major rivers deeply notch the caprock, as exemplified by Palo Duro Canyon and Caprock Canyons State Parks.

On the High Plains, widespread small, intermittent streams dominate the drainage. The Canadian River cuts across the province, creating the Canadian Breaks and separating the Central High Plains from the Southern High Plains. Pecos River drainage erodes the west-facing escarpment of the Southern High Plains, which terminates against the Edwards Plateau on the south.

Basin and Range. The Basin and Range province contains eight mountain peaks that are higher than 8,000 feet. At 8,749 feet, Guadalupe Peak is the highest point in Texas. Mountain ranges generally trend nearly north-south and rise abruptly from barren rocky plains.

Plateaus in which the rocks are nearly horizontal and less deformed commonly flank the mountains. Cores of strongly folded and faulted sedimentary and volcanic rocks or of granite rocks compose the interiors of mountain ranges. Volcanic rocks form many peaks. Large flows of volcanic ash and thick deposits of volcanic debris flank the slopes of most former volcanoes. Ancient volcanic activity of the Texas Basin and Range province was mostly explosive in nature, like Mount Saint Helens. Volcanoes that poured successive lava flows are uncommon. Eroded craters, where the cores of volcanoes collapsed and subsided, are abundant.

Gray oak, pinyon pine, and alligator juniper parks drape the highest elevations. Creosote bush and lechuguilla shrubs sparsely populate plateaus and intermediate elevations. Tobosa black grama grassland occupies the low basins.

Floodplain Plant Communities – Floodplain areas are a microclimatic area that results in a characteristic plant community that is similar in all three physiographic provinces. Floodplain plant communities are an important reference community since many storm water practices are located with this area. Floodplains occur along streams in both steep and level areas. The most noteworthy plants found along floodplains are River Birch, Willows, Poplars, Maple, Sweet Gum, Sycamore, Box Elder, Green Ash, American Elm, Swamp White Oak, Bur Oak, Honeylocust and Hackberry. Shrubs commonly found in floodplains include Shrub Willows, Yaupon, Buttonbush, Blackberry, and Elderberry.

F.3.3 Other Considerations in Plant Selection

Use or Function

In selecting plants, consideration must be given to their desired function in the storm water management facility. Is the plant needed as ground cover, soil stabilizer, biofilter or source of shade? Will the plant be placed for functional or aesthetic purposes? Does the adjacent use provide conflicts or potential problems and require a barrier, screen, or buffer? Nearly every plant and plant location should be provided to serve some function in addition to any aesthetic appeal.

Plant Characteristics

Certain plant characteristics are so obvious, they may actually be overlooked in the plant selection. These are:

- Size
- Shape

For example, tree limbs, after several years, can grow into power lines. A wide growing shrub may block maintenance access to a storm water facility. Consider how these characteristics can work for you or against you, today and in the future.

Other plant characteristics must be considered to determine how the plant grows and functions seasonally, and whether the plant will meet the needs of the facility today and in the future. Some of these characteristics are:

- Growth Rate
- Regeneration Capacity
- Maintenance Requirements (e.g. mowing, harvesting, leaf collection, etc.)
- Aesthetics

In urban or suburban settings, a plant's aesthetic interest may be of greater importance. Residents living next to a storm water system may desire that the facility be appealing or interesting to look at throughout the year. Aesthetics is an important factor to consider in the design of these systems. Failure to consider the aesthetic appeal of a facility to the surrounding residents may result in reduced value to nearby lots. Careful attention to the design and planting of a facility can result in maintained or increased values of a property.

Availability and Cost

Often overlooked in plant selection is the availability from wholesalers and the cost of the plant material. There are many plants listed in landscape books that are not readily available from the nurseries. Without knowledge of what is available, time spent researching and finding the one plant that meets all the needs will be wasted, if it is not available from the growers. It may require shipping, therefore, making it more costly than the budget may allow. Some planting requirements, however, may require a special effort to find the specific plant that fulfills the needs of the site and the function of the plant in the landscape.

Native versus Nonnative Species

This Manual encourages the use of native plants in storm water management facilities, since they are best suited to thrive under the physiographic and hardiness conditions encountered at a site. Unfortunately, not all native plants provide the desired landscape or appearance, and may not always be available in quantity from local nurseries. Therefore, naturalized plants that are not native species, but can thrive and reproduce in the new area may be a useful alternative.

Because all landscaping needs may not be met by native or naturalized plants, some ornamental and exotic species are provided in this guide that can survive under difficult conditions encountered in a storm water management facility. Since many storm water facilities are adjacent to residential areas, the objectives of the storm water planting plan may shift to resemble the more controlled appearance of nearby yards, or to provide a pleasing view. Great care should be taken; however, when introducing plant species so as not to create a situation where they may become invasive and take over adjacent natural plant communities.

Moisture Status

In landscaping storm water management facilities, hydrology plays a large role in determining which species will survive in a given location.

For areas that are to be planted within a storm water management facility it is necessary to determine what type of hydrologic zones will be created within the facility.

The six zones shown in Table F-1 in the next section describe the different conditions encountered in storm water management facilities. Every facility does not necessarily reflect all of these zones. The hydrologic zones designate the degree of tolerance the plant exhibits to differing degrees of inundation by

water. Each zone has its own set of plant selection criteria based on the hydrology of the zone, the storm water functions required of the plant and the desired landscape effect.

F.4 Specific Landscaping Criteria for Structural Storm Water Controls

F.4.1 Storm Water Ponds and Wetlands

Storm water ponds and wetlands are engineered basins and wetland areas designed to control and treat storm water runoff. Aquatic vegetation plays an important role in pollutant removal in both storm water ponds and wetlands. In addition, vegetation can enhance the appearance of a pond or wetland, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris.

Within a storm water pond or wetland, there are various hydrologic zones as shown in Table F-1 that must be considered in plant selection. These hydrologic zones designate the degree of tolerance a plant must have to differing degrees of inundation by water. Hydrologic conditions in an area may fluctuate in unpredictable ways; thus the use of plants capable of tolerating wide varieties of hydrologic conditions greatly increases the successful establishment of a planting. Plants suited for specific hydrologic conditions may perish when those conditions change, exposing the soil, and therefore, increasing the chance for erosion. Each of the hydrologic zones is described in more detail below along with examples of appropriate plant species.

Table F-1 Hydrologic Zones

Zone #	Zone Description	Hydrologic Conditions
Zone 1	Deep Water Pool	1-6 feet depth (permanent pool)
Zone 2	Shallow Water Bench	Normal pool elevation to 1 foot depth
Zone 3	Shoreline Fringe	Regularly inundated
Zone 4	Riparian Fringe	Periodically inundated
Zone 5	Floodplain Terrace	Infrequently inundated
Zone 6	Upland Slopes	Seldom or never inundated

Zone 1: Deep Water Area *(1- 6 Feet)*

Ponds and wetlands both have deep pool areas that comprise Zone 1. These pools range from one to six feet in depth, and are best colonized by submergent plants, if at all.

This pondscaping zone is *not* routinely planted for several reasons. First, the availability of plant materials that can survive and grow in this zone is limited, and it is also feared that plants could clog the storm water facility outlet structure. In many cases, these plants will gradually become established through natural recolonization (e.g., transport of plant fragments from other ponds via the feet and legs of waterfowl). If submerged plant material is commercially available and clogging concerns are addressed, this area can be planted. The function of the planting is to reduce resedimentation and improve oxidation while creating a greater aquatic habitat.

- Plant material must be able to withstand constant inundation of water of one foot or greater in depth.

- Plants may be submerged partially or entirely.
- Plants should be able to enhance pollutant uptake.
- Plants may provide food and cover for waterfowl, desirable insects, and other aquatic life.

Some suggested emergent or submergent species include, but are not limited to: Water Lily, Deepwater Duck Potato, Spatterdock, Wild Celery and Redhead Grass.

Zone 2: Shallow Water Bench *(Normal Pool To 1 Foot)*

Zone 2 includes all areas that are inundated below the normal pool to a depth of one foot, and is the primary area where emergent plants will grow in storm water wetlands. Zone 2 also coincides with the aquatic bench found in storm water ponds. This zone offers ideal conditions for the growth of many emergent wetland species. These areas may be located at the edge of the pond or on low mounds of earth located below the surface of the water within the pond. When planted, Zone 2 can be an important habitat for many aquatic and nonaquatic animals, creating a diverse food chain. This food chain includes predators, allowing a natural regulation of mosquito populations, thereby reducing the need for insecticidal applications.

- Plant material must be able to withstand constant inundation of water to depths between six inches and one foot deep.
- Plants will be partially submerged.
- Plants should be able to enhance pollutant uptake.
- Plants may provide food and cover for waterfowl, desirable insects and other aquatic life.

Common emergent wetland plant species used for storm water wetlands and on the aquatic benches of storm water ponds include, but are not limited to: Arrowhead/Duck Potato, Soft Rush, various Sedges, Softstem Bulrush, Switchgrass, Pickerelweed, Pond Cypress and various Asters.

Zone 3: Shoreline Fringe *(Regularly Inundated)*

Zone 3 encompasses the shoreline of a pond or wetland, and extends vertically about one foot in elevation from the normal pool. This zone includes the safety bench of a pond, and may also be periodically inundated if storm events are subject to extended detention. This zone occurs in a wet pond or shallow marsh and can be the most difficult to establish since plants must be able to withstand inundation of water during storms, when wind might blow water into the area, or the occasional drought during the summer. In order to stabilize the soil in this zone, Zone 3 must have a vigorous cover.

- Plants should stabilize the shoreline to minimize erosion caused by wave and wind action or water fluctuation.
- Plant material must be able to withstand occasional inundation of water. Plants will be partially submerged partially at this time.
- Plant material should, whenever possible, shade the shoreline, especially the southern exposure. This will help to reduce the water temperature.
- Plants should be able to enhance pollutant uptake.
- Plants may provide food and cover for waterfowl, songbirds, and wildlife. Plants could also be selected and located to control overpopulation of waterfowl.
- Plants should be located to reduce human access, where there are potential hazards, but should not block the maintenance access.
- Plants should have very low maintenance requirements, since they may be difficult or impossible to reach.

- Plants should be resistant to disease and other problems which require chemical applications (since chemical application is not advised in storm water ponds).

Many of the emergent wetland plants that perform well in Zone 2 also thrive in Zone 3. Some other species that do well include Broom Grass, Upland Sea-Oats, Dwarf Tickseed, various Ferns, Hawthorns. If shading is needed along the shoreline, the following tree species are suggested: Boxelder, Ash, Willow, Red Maples and Willow Oak.

Zone 4: Riparian Fringe *(Periodically Inundated)*

Zone 4 extends from one to four feet in elevation above the normal pool. Plants in this zone are subject to periodic inundation after storms, and may experience saturated or partly saturated soil inundation. Nearly all of the temporary extended detention (ED) storage area is included within this zone.

- Plants must be able to withstand periodic inundation of water after storms, as well as occasional drought during the warm summer months.
- Plants should stabilize the ground from erosion caused by run-off.
- Plants should shade the low flow channel to reduce the pool warming whenever possible.
- Plants should be able to enhance pollutant uptake.
- Plant material should have very low maintenance, since they may be difficult or impossible to access.
- Plants may provide food and cover for waterfowl, songbirds and wildlife. Plants may also be selected and located to control overpopulation of waterfowl.
- Plants should be located to reduce pedestrian access to the deeper pools.

Some frequently used plant species in Zone 4 include Broom Grass, Yellow Indian Grass, Joe Pye Weed, Lilies, Flatsedge, Hollies, Forsythia, Lovegrass, Hawthorn and Sugar Maples.

Zone 5: Floodplain Terrace *(Infrequently Inundated)*

Zone 5 is periodically inundated by flood waters that quickly recede in a day or less. Operationally, Zone 5 extends from the maximum two year or SP_v water surface elevation up to the 25 or 100 year maximum water surface elevation. Key landscaping objectives for Zone 5 are to stabilize the steep slopes characteristic of this zone, and establish a low maintenance, natural vegetation.

- Plant material should be able to withstand occasional but brief inundation during storms, although typical moisture conditions may be moist, slightly wet, or even swing entirely to drought conditions during the dry weather periods.
- Plants should stabilize the basin slopes from erosion.
- Ground cover should be very low maintenance, since they may be difficult to access on steep slopes or if the frequency of mowing is limited. A dense tree cover may help reduce maintenance and discourage resident geese.
- Plants may provide food and cover for waterfowl, songbirds, and wildlife.
- Placement of plant material in Zone 5 is often critical, as it often creates a visual focal point and provides structure and shade for a greater variety of plants.

Some commonly planted species in Zone 5 include many wildflowers or native grasses, many Fescues, many Viburnums, Witch Hazel, Blueberry, American Holly, American Elderberry and Red Oak.

Zone 6: Upland Slopes *(Seldom or Never Inundated)*

The last zone extends above the maximum 100 year water surface elevation, and often includes the outer buffer of a pond or wetland. Unlike other zones, this upland area may have sidewalks, bike paths, retaining walls, and maintenance access roads. Care should be taken to locate plants so they will not overgrow these routes or create hiding places that might make the area unsafe.

- Plant material is capable of surviving the particular conditions of the site. Thus, it is not necessary to select plant material that will tolerate any inundation. Rather, plant selections should be made based on soil condition, light, and function within the landscape.
- Ground covers should emphasize infrequent mowing to reduce the cost of maintaining this landscape.

Placement of plants in Zone 6 is important since they are often used to create a visual focal point, frame a desirable view, screen undesirable views, or serve as a buffer.

Some frequently used plant species in Zone 6 include most ornamentals (as long as soils drain well, many wildflowers or native grasses, Linden, False Cypress, Magnolia, most Spruce, Mountain Ash and most Pine.

- Table F-2 provides a list of selected wetland plants for storm water ponds and wetlands. For hydrologic zones 1-4, provide shade to allow a greater variety of plant materials. Particular attention should be paid to seasonal color and texture of these plantings.

Table F-2 Wetland Plants (Herbaceous Species) for Storm Water Facilities

Scientific Name	Common Name	Hydrologic Zone
Acorus calumus	Sweetflag	2
Andropogon gerardii	Big Bluestem	6
Andropogon glomeratus	Bushy Broom Grass	3
Andropogon virginicus	Broom Grass	4
Asclepias tuberosa	Butterfly-weed	6
Bouteloua certipendula	Sideoats Grama	6
Buchloe dactyliodes	Buffalograss	6
Carex spp.	Caric Sedges	2
Chasmanthium latifolium	Upland Sea-Oats	3
Coreopsis tinctoria	Dwarf Tickseed	3
Cynodon dactylon	Bermuda Grass	5,6
Echinacea purpurea	Purple Coneflower	6
Elocharis quadrangulata	Square Stem Spikerush	2
Elymus Canadensis	Canada Wildrye	4,5
Elymus virginicus	Virginia Wildrye	4,5
Eupatorium fistolosum	Joe Pye Weed	4
Euptorium serotinum	Late Boneset	3,4

Scientific Name	Common Name	Hydrologic Zone
Eustoma grandiflora	Texas Bluebells	4
Helianthus angustifolius	Swamp Sunflower	2
Helianthus maximiliani	Maximilian Sunflower	3,4,5,6
Hibiscus laevis	Halberdleaf Hibiscus	2,3
Juncus effuses	Soft Rush	2
Leersia oryzoides	Rice Cut Grass	2
Leptochola dubia	Green Spangletop	6
Liatris mucronata	Gayfeather	6
Liatris punctata	Gayfeather	6
Liatris pycnostachya	Gayfeather	5,6
Liatris spicata	Spiked Gayfeather	3
Lobelia cardinalis	Cardinal Flower	3
Malvaviscus drummondii	Turk's Cap	4,5,6
Nuphar luteum	Spatterdock	1
Nymphaea mexicana	Yellow Water Lily	1
Nymphaea odorata	Fragrant Water Lily	1
Osmunda cinnamomea	Cinnamon Fern	3
Osmunda regalis	Royal Fern	3
Panicum capillare	Witchgrass	3,4,5,6
Panicum virgatum	Switchgrass	2
Peltandra virginica	Green Arum	2
Pennisetum alopecuroides	Fountaingrass	6
Poa arachnifera	Texas Bluegrass	6
Polygonum hydropiperoides	Smartweed	2
Pontederia cordata	Pickerelweed	2,3
Pontederia lanceolata	Pickerelweed	2
Rudbeckia hirta	Black-eyed Susan	4
Sagittaria lancifolia	Lance-leaf Arrowhead	2
Sagittaria latifolia	Duck Potato	2
Salvia farinacea	Mealy Blue Sage	6
Salvia greggii	Autumn Sage	6
Saururus cernuus	Lizard's Tail	2
Schizachyrium scoparium	Little Bluestem	6

Scientific Name	Common Name	Hydrologic Zone
Scirpus americanus	Three-square	2
Scirpus californicus	Giant Bulrush	2
Scirpus validus	Softstem Bulrush	2,3
Sorgham nutans	Yellow Indian Grass	4
Tripsacum dactyloides	Eastern Gammagrass	3,4,5,6
Valpia octoflora	Common Sixweeksgrass	6
Woodwardia virginica	Virginia Chain Fern	2

Source: Aquascape, Inc.
Texas Parks and Wildlife Department

Zone 1: 12 to 36 inch depth below normal pool elevation
Water Lily, Deep Water Duck Potato, Spatterdock, Wild Celery, Redhead Grass

Zone 2: 0 to 12 inch depth below normal pool elevation
Arrowhead/Duck Potato, Soft Rush, various Sedges, Softstem Bulrush, Switchgrass, Southern Blue Flag Iris, Swamp Hibiscus, Swamp Lily, Pickerelweed, Pond Cypress, various Asters

Zone 3: 0 to 12 inch elevation above normal pool elevation
Various species from above, Broom Grass, Upland Sea-Oats, Dwarf Tickseed, various Ferns, Hawthorns, Boxelder, Ash, Willow, Red Maple, Willow Oak

Zone 4: 1 to 4 foot elevation above normal pool elevation
Broom Grass, Yellow Indian Grass, Ironweed, Joe Pye Weed, various Lilies, Flatsedge, Hollies, Lovegrass, Hawthorn, Sugar Maple

Zone 5: SP_v to Q_p or Q_f water surface elevation
Many wildflowers or native grasses, many Fescues, many Viburnums, Witch Hazel, Blueberry, American Holly, American Elderberry, Red Oak

Zone 6: Q_f water surface elevation and above
Many ornamentals as long as soils drain well, many wildflowers or native grasses, Linden, False Cypress, Magnolia, most Spruce, Mountain Ash, most Pine

Figure F-3 Legend of Hydrologic Zones Around Storm Water Facilities

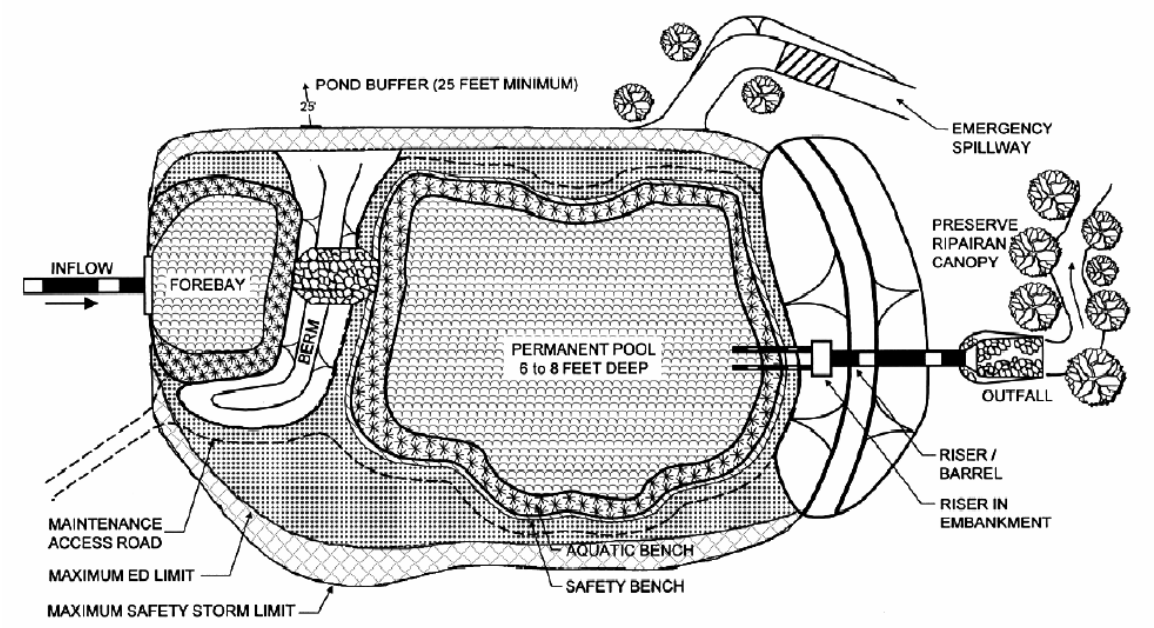


Figure F-4 Plan View of Hydrologic Zones around Storm Water Wet ED Pond

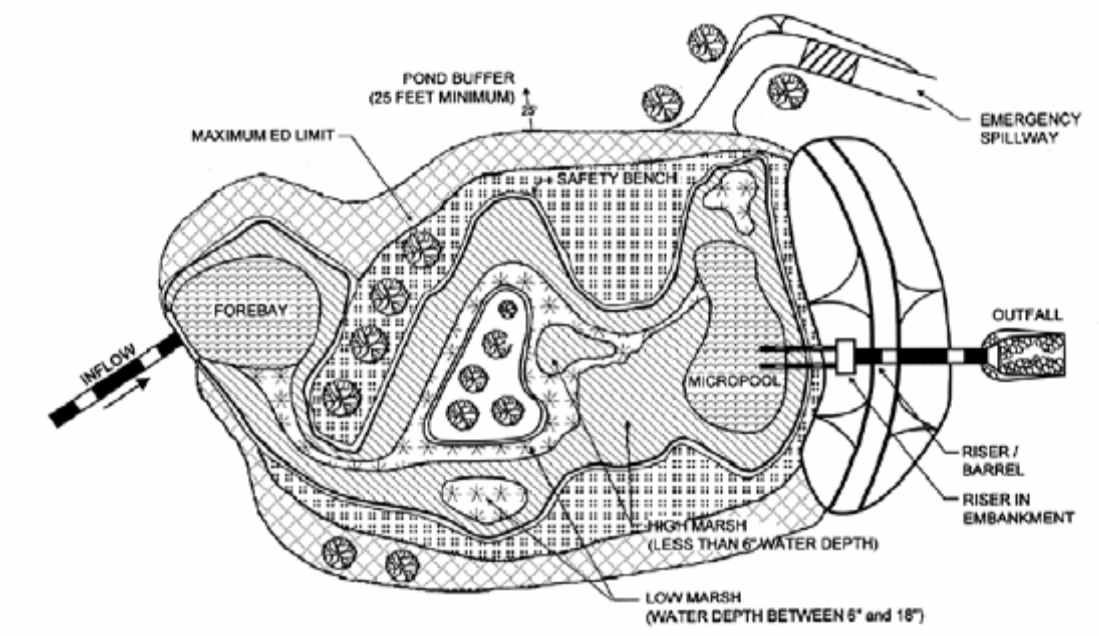


Figure F-5 Plan View of Hydrologic Zones around Storm Water ED Shallow Wetland

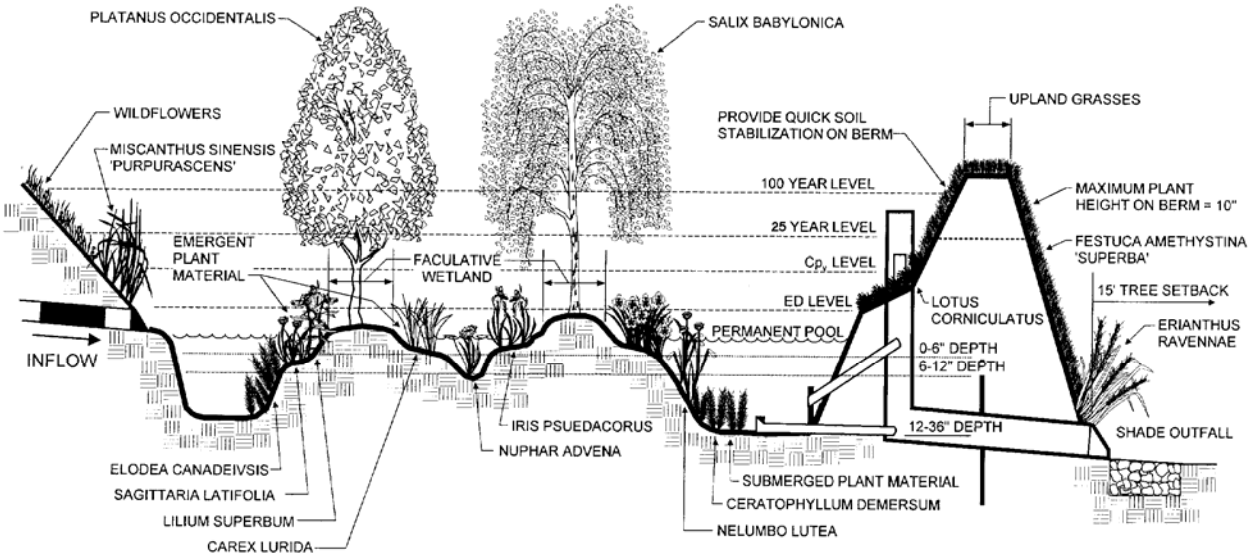


Figure F-6 Section of Typical Shallow ED Wetland

F.4.2 Bioretention Areas

Bioretention areas are structural storm water controls that capture and treat runoff using soils and vegetation in shallow basins or landscaped areas. Landscaping is therefore critical to the performance and function of these facilities. Below are guidelines for soil characteristics, mulching, and plant selection for bioretention areas.

Planting Soil Bed Characteristics

The characteristics of the soil for the bioretention facility are perhaps as important as the facility location and size. The soil must be permeable enough to allow runoff to filter through the media, while having characteristics suitable to promote and sustain a robust vegetative cover crop. In addition, much of the nutrient pollutant uptake (nitrogen and phosphorus) is accomplished through adsorption and microbial activity within the soil profile. Therefore, the soils must balance soil chemistry and physical properties to support biotic communities above and below ground.

The planting soil should be a sandy loam, loamy sand, loam, or a loam/sand mix (should contain a minimum of 35 to 60% sand, by volume). The clay content for these soils should by less than 25% by volume. Soils should fall within the SM, ML, SC classifications of the Unified Soil Classification System (USCS). A permeability of at least 1.0 feet per day (0.5"/hr) is required (a conservative value of 0.5 feet per day should be used for design). The soil should be free of stones, stumps, roots, or other woody material over 1" in diameter. Brush or seeds from noxious weeds, such as Johnson Grass, Mugwort, Nutsedge, and Canadian Thistle should not be present in the soils. Placement of the planting soil should be in lifts of 12 to 18", loosely compacted (tamped lightly with a dozer or backhoe bucket). The specific characteristics are presented in Table F-3.

Table F-3 Planting Soil Characteristics

Parameter	Value
pH range	5.2 to 7.00
Organic matter	1.5 to 4.0%
Magnesium	35 lbs. per acre, minimum (0.0072 lbs/Sq yd)
Phosphorus (P_2O_5)	75 lbs. per acre, minimum (0.0154 lbs/Sq yd)
Potassium (K_2O)	85lbs. per acre, minimum (0.0175 lbs/Sq yd)
Soluble salts	500 ppm
Clay	10 to 25%
Silt	30 to 55%
Sand	35 to 60%

(Adapted from EQR, 1996; ETAB, 1993)

Mulch Layer

The mulch layer plays an important role in the performance of the bioretention system. The mulch layer helps maintain soil moisture and avoids surface sealing which reduces permeability. Mulch helps prevent erosion, and provides a micro-environment suitable for soil biota at the mulch/soil interface. It also serves as a pretreatment layer, trapping the finer sediments which remain suspended after the primary pretreatment. The mulch layer should be standard landscape style, single or double, shredded hardwood mulch or chips. The mulch layer should be well aged (stockpiled or stored for at least 12 months), uniform in color, and free of other materials, such as weed seeds, soil, roots, etc. The mulch should be applied to a maximum depth of three inches. Grass clippings should not be used as a mulch material.

Planting Plan Guidance

Plant material selection should be based on the goal of simulating a terrestrial forested community of native species. Bioretention simulates an ecosystem consisting of an upland-oriented community dominated by trees, but having a distinct community, or sub-canopy, of understory trees, shrubs and herbaceous materials. The intent is to establish a diverse, dense plant cover to treat storm water runoff and withstand urban stresses from insect and disease infestations, drought, temperature, wind, and exposure.

The proper selection and installation of plant materials is key to a successful system. There are essentially three zones within a bioretention facility (Figure F-7). The lowest elevation supports plant species adapted to standing and fluctuating water levels. The middle elevation supports a slightly drier group of plants, but still tolerates fluctuating water levels. The outer edge is the highest elevation and generally supports plants adapted to dryer conditions. A sample of appropriate plant materials for bioretention facilities are included in Table F-4. More potential bioretention species can be found in the wetland plant list in subsection F.5.

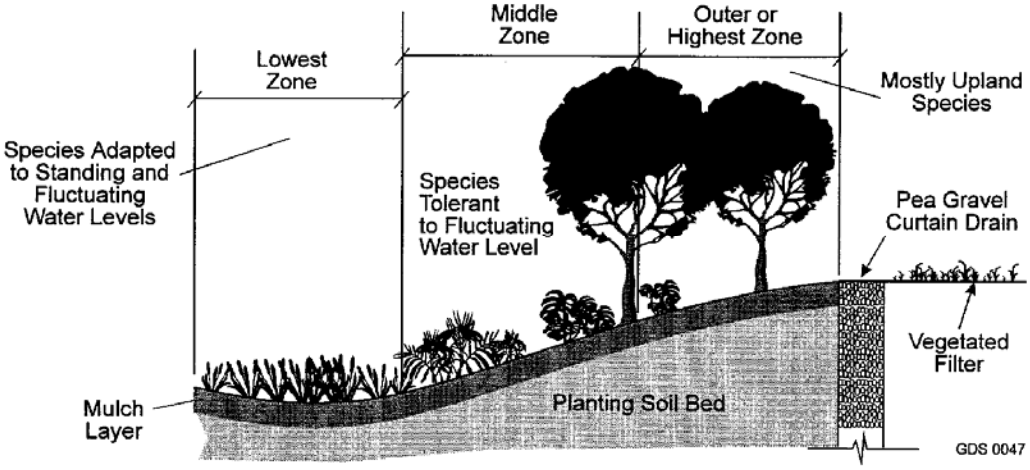


Figure F-7 Planting Zones for Bioretention Facilities

The layout of plant material should be flexible, but should follow the general principals described below. The objective is to have a system that resembles a random and natural plant layout, while maintaining optimal conditions for plant establishment and growth.

- Native plant species should be specified over exotic or foreign species.
- Appropriate vegetation should be selected based on the zone of hydric tolerance
- Species layout should generally be random and natural.

The tree-to-shrub ratio should be 2:1 to 3:1. On average, the trees should be spaced 8 feet apart.
Plants should be placed at irregular intervals to replicate a natural forest.
Woody vegetation should not be specified at inflow locations.

A canopy should be established with an understory of shrubs and herbaceous materials.

- Woody vegetation should not be specified in the vicinity of inflow locations.
- Trees should be planted primarily along the perimeter of the bioretention area.
- Urban stressors (e.g., wind, sun, exposure, insect and disease infestation, drought) should be

considered when laying out the planting plan.

- Noxious weeds should not be specified.
- Aesthetics and visual characteristics should be a prime consideration.
- Traffic and safety issues must be considered.
- Existing and proposed utilities must be identified and considered.

Plant materials should conform to the American Standard Nursery Stock, published by the American Association of Nurserymen, and should be selected from certified, reputable nurseries. Planting specifications should be prepared by the designer and should include a sequence of construction, a description of the contractor's responsibilities, a planting schedule and installation specifications, initial maintenance, and a warranty period and expectations of plant survival. Table F-5 presents some typical issues for planting specifications. Figure F-8 shows an example of a sample planting plan for a bioretention area.

Table F-4 Commonly Used Species for Bioretention Areas

Trees	Shrubs	Herbaceous Species
Acer rubrum Red Maple (Zones 2, 3, 4)	*Amorpha fruticosa* False Indigo (Zones 3, 4)	*Andropogon virginicus* Broom Sedge/ Grass (Zone 4)
Betula nigra River Birch (Zones 4, 5)	*Aronia arbutifolia* Red Chokeberry (Zones 2, 3)	*Eupatorium fistolosum* Joe Pye Weed (Zone 4)
Cercis canadensis Eastern Redbud (Zones 4, 5)	*Callicarpa Americana* American Beautyberry (Zones 4, 5)	*Iris pseudacorus* Yellow Iris
Crataegus reverchonii Reverchon's Hawthorn (Zone 6)	*Hamemelis virginiana* Witch Hazel (Zone 5)	*Lobelia cardinalis* Cardinal Flower (Zone 3)
Juglans nigra Black Walnut (Zone 6)	*Lindera benzoin* Spicebush	*Malvaviscus drummondii* Turk's Cap (Zones 4, 5, 6)
Juniperus virginiana Eastern Red Cedar (Zones 5, 6)	*Myrica pennsylvanica* Bayberry	*Panicum capillare* Witchgrass (Zones 3, 4, 5, 6)
Platanus occidentalis Sycamore	*Prunus mexicana* Mexican Plum (Zones 5, 6)	*Panicum virgatum* Switchgrass (Zone 2)
Quercus phellos Willow Oak (Zones 3, 4, 5)	*Rhamnus caroliniana* Carolina Buckthorn (Zones 4, 5, 6)	*Pennisetum alopecuroides* Fountaingrass (Zone 6)
Quercus macrocarpa Bur Oak (Zones 5, 6)	*Viburnum rufidumlum* Rusty Blackhaw (Zones 4, 5, 6)	*Rudbeckia hirta* Black Eyed Susan (Zone 4)

Table F-5 Planting Plan Specification Issues for Bioretention Areas

Specification Element	Elements
Sequence of Construction	Describe site preparation activities, soil amendments, etc.; address erosion and sediment control procedures; specify step-by-step procedure for plant installation.
Contractor's Responsibilities	Specify the contractors responsibilities, such as watering, care of plant material during transport, timeliness of installation, repairs due to vandalism, etc.
Planting Schedule and Specifications	Specify the materials to be installed, the type of materials (e.g., B&B, bare root, containerized); time of year of installations, sequence of installation of types of plants; fertilization, stabilization seeding, if required; watering and general care.
Maintenance	Specify inspection periods; mulching frequency; removal and replacement of dead and diseased vegetation; treatment of diseased trees; watering schedule after initial installation (once per day for 14 days is common); repair and replacement of staking and wires.
Warranty	Specify warranty period, the required survival rate, and expected condition of plant species at the end of the warranty.

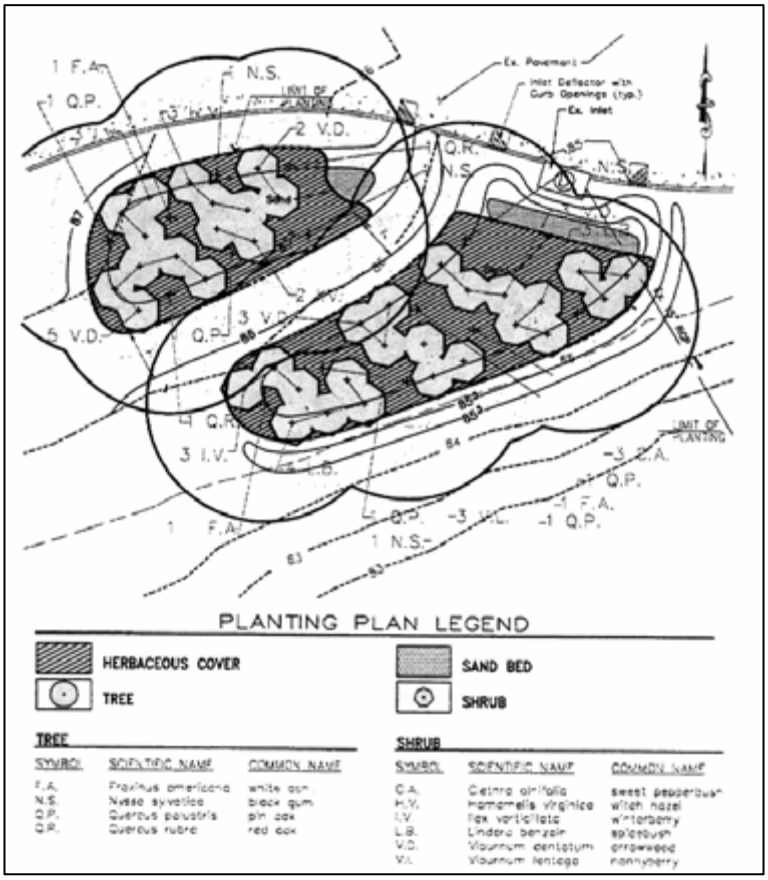


Figure F-8 Sample Bioretention Area Planting Plan
(Source: VDCR, 1999)

F.4.3 Surface Sand Filters and Infiltration Trenches

Both surface sand filters and infiltration trenches can be designed with a grass cover to aid in pollutant removal and prevent clogging. The sand filter or trench is covered with permeable topsoil and planted with grass in a landscaped area. Properly planted, these facilities can be designed to blend into natural surroundings.

Grass should be capable of withstanding frequent periods of inundation and drought. Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible.

Design Constraints:

- Check with your local review authority to see if the planning of a grass cover or turf over a sand filter or infiltration trench is allowed.
- Do not plant trees or provide shade within 15 feet of infiltration or filtering area or where leaf litter will collect and clog infiltration area.
- Do not locate plants to block maintenance access to the facility.
- Sod areas with heavy flows that are not stabilized with erosion control mats.
- Divert flows temporarily from seeded areas until stabilized.
- Planting on any area requiring a filter fabric should include material selected with care to insure that no tap roots will penetrate the filter fabric.

F.4.4 Enhanced Swales, Grass Channels and Filter Strips

Table F-6 provides a number of grass species that perform well in the stressful environment of an open channel structural control such as an enhanced swale or grass channel, or for grass filter strips. In addition, wet swales may include other wetland species (see F.4.1). Select plant material capable of salt tolerance in areas that may include high salt levels.

Table F.6 Common Grass Species for Dry and Wet Swales and Grass Channels

Common Name	Scientific Name	Notes
Bermuda grass	*Cynodon dactylon*	1,2
Big Bluestem	*Andropogon gerardii*	2, 3, Not for wet swales
Witchgrass	*Panicum capillare*	2,3, Not for wet swales
Switchgrass	*Panicum virgatum*	3
Buffalograss	*Buchloe dactyloides*	1, 2, 3
Bushy Bluestem	*Andropogon glomeratus*	2,3
Virginia Wildrye	*Elymus virginicus*	2,3,4 Not for wet swales
Texas Bluegrass	*Poa arachnifera*	2,3, Not for wet swales
Common Sixweeksgrass	*Vulpia octoflora*	2,3
Green Sprangletop	*Leptochloa dubia*	2,3
Canada Wildrye	*Elymus canadensis*	2,3,4, Wet swales

Common Name	Scientific Name	Notes
Longleaf Chasmanthium / Upland Sea Oats	*Chasmanthium latifolium*	2,3,4
Eastern Gammagrass	*Tripsacum dactyloides*	2,3

Note 1: These grasses are sod-forming and can withstand frequent inundation, and are thus ideal for the swale or grass channel environment. Most are salt-tolerant, as well.
Note 2: Where possible, one or more of these grasses should be in the seed mixes
Note 3: Native Texas grasses
Note 4: Shade tolerant

F.4.5 Green Roofs

- The growth medium is generally 2 to 6 inches thick and made of a material that drains relatively quickly. Commercial mixtures containing coir (coconut fiber), pumice, or expanded clay are available. Sand, gravel, crushed brick, and peat are also commonly used. Suppliers recommend limiting organic material to less than 33% to reduce fire hazards. The City of Portland, Oregon has found a mix of 1/3 topsoil, 1/3 compost, and 1/3 perlite may be sufficient for many applications. Growth media can weigh from 16 to 35 psf when saturated depending on the type (intensive/extensive), with the most typical range being from 10-25 psf.
- When dry, all of the growth media are light-weight and prone to wind erosion. It is important to keep media covered before planting and ensure good coverage after vegetation is established.
- Selecting the right vegetation is critical to minimize maintenance requirements. Due to the shallowness of the growing medium and the extreme desert-like microclimate on many roofs, plants are typically alpine, dryland, or indigenous. Ideally, the vegetation should be:
  - Drought-tolerant, requiring little or no irrigation after establishment
  - Self-sustaining, without fertilizers, pesticides, or herbicides
  - Able to withstand heat, cold, and high winds
  - Shallow root structure
  - Low growing, needing little or no mowing or trimming
  - Fire resistant
  - Perennial or self propagating, able to spread and cover blank spots by itself

  Visit www.txsmartscape.com to look up plants meeting the above criteria.
- A mix of sedum/succulent plant communities is recommended because they possess many of these attributes. Certain wildflowers, herbs, forbs, grasses, mosses, and other low groundcovers can also be used to provide additional habitat benefits or aesthetics; however, these plants need more watering and maintenance to survive and keep their appearance.
- Green roof vegetation is usually established by one or more of the following methods: seeding, cuttings, vegetation mats, and plugs/potted plants.
  - Seeds can be either hand sown or broadcast in a slurry (hydraseeded). Seeding takes longer to establish and requires more weeding, erosion control, and watering than the other methods.
  - Cuttings or sprigs are small plant sections. They are hand sown and require more weeding, erosion control, and watering than mats.
  - Vegetation mats are sod-like mats that achieve full plant coverage very quickly. They provide immediate erosion control, do not need mulch, and minimize weed intrusion. They generally require less ongoing maintenance than the other methods.
  - Plugs or potted plants may provide more design flexibility than mats. However, they take longer to achieve full coverage, are more prone to erosion, need more watering during establishment,

require mulching, and more weeding.

- Green roof vegetation is most easily established during the spring or fall.

F.5 Trees and Shrubs for Storm Water Facilities

The following pages present a detailed list of wetland trees and shrubs that may be used for storm water management facilities such as storm water ponds, storm water wetlands and bioretention areas. (Source: Garber and Moorhead, 1999)

Table F-7 Wetland indicator status, growth form, flood tolerance and seed dispersal and treatment for selected native wetland trees and shrubs.

Species	Indicator*	Form	Flood Tolerance**	Seed Dispersal***	Seed Treatments****	Comments
Boxelder *Acer negundo*	FACW-	Tree	T	Sept.-Mar.	Cold Strat. 30-40 Days (Mech. Rup. Peri-carp)	Can propagate by softwood cuttings
Red Maple *Acer rubrum*	FAC	Tree	T	Apr.-July	Strat. not required	Can propagate by softwood cuttings, tissue culture
Hazel Alder *Alnus serrulata*	OBL	Tree	NE	Sept.-Oct.	Cold Strat. 30-60 Days	Can propagate by cuttings, tissue culture
Common Pawpaw *Asimina triloba*	FAC-	Tree	I	Sept.-Oct.	Scarification Re-quired Cold Strat. 60-90 Days	
River Birch *Betula nigra*	FACW	Tree	IT	May-June	Cold Strat. 60-90 Days	Can propagate by softwood cuttings
American Hornbeam *Carpinus caroliniana*	FAC	Tree	WT	Oct.-Spring	Cold Strat. 60 Days	
Water Hickory C *Carya aquatica*	OBL	Tree	IT	Oct.-Dec.	Cold Strat. 30-90 Days Warm Strat. 60 Days	
Bitternut Hickory *I Carya cordiformus*	FAC	Tree	NE	Sept.-Dec.	Cold Strat. 90 Days	
Pecan *Carya illinoensis*	FAC +	Tree	IT	Sept.-Dec.	Cold Strat. 30-90 Days	
Sugarberry *Celtis laevigata*	FAC	Tree	IT	Oct.-Dec.	Cold Strat. 60-90 Days	
Common Buttonbush *Cephalanthus occidentalis*	OBL	Shrub	VT	Sept.-Oct.	Strat. not req.	
American Sycamore *Platanus occidentalis*	FAC +	Tree	T	Feb.-Apr.	Cold Strat. *60-90* Days	
Eastern Cottonwood *Populus deltoides*	FAC	Tree	VT	May-Aug.	Strat. not req.	Can propagate by cuttings
Wafer Ash *Ptelea trifoliata*	FAC	Shrub	NE	Sept.	Cold Strat. *90-120* Days	

Species	Indicator*	Form	Flood Tolerance**	Seed Dispersal***	Seed Treatments****	Comments
Cherrybark Oak *Quercus pagoda*	FAC +	Tree	I	Aug.-Dec.Cold Strat.	*30-90* Days	Red Oak group
Laurel Oak *Quercus laurifolia*	FACW	Tree	IT	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Overcup Oak *Quercus lyrata*	OBL	Tree	T	Aug.-Dec.	Strat. not req.	White Oak group
Water Oak *Quercus nigra*	FAC+	Tree	T	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Willow Oak *Quercus phellos*	FACW	Tree	T	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Shumard Oak *Quercus shumardii*	FAC	Tree	IT	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Black Willow *Salix nigra*	FACW+	Tree	VT	June-July	Not required.	Seed will not remain viable in storage. Plant within 10 days after collection.Can propagate by cuttings
Baldcypress *Taxodium distichum* var. *distichum*	OBL	Tree	VT	Oct.-Nov.	Cold Strat. *90* Days.	Soak seed for S min. in ethyl alcohol be-fore placing in cold stratification.
Pondcypress *Taxodium distichum* var. *nutans*	OBL	Tree	VT	Oct.-Nov.	Cold Strat. *60-90* Days.	Soak seed for *24* to *48* hrs. in 0.0196 cit-ric acid before plac-ing in cold stratification.
American Elm *Ulmus Americana*	FAC	Tree	T	Mar.-June	Cold Strat. 60-90 Days	Can propagate by cuttings
Cedar Elm *Ulmus crassifolia*	FAC	Tree	I	Apr.-June	Cold Strat. 60-90 Days	Can propagate by cuttings
Slippery Elm *Ulmus rubra*	FAC	Tree	I	Apr.-June	Cold Strat. 60-90 Days	Can propagate by cuttings
Rough-Leaf Dogwood *Cornus drummondii*	FAC	Tree	T	Aug.-Jan.	Warm Strat. 70°-80° 1 Day Cold Strat. 30 Days	
Hawthornes *Crataegus reverchonii* *Crataegus viridis*	FAC	Shrub	IT	Fall-Winter	May Req. Scari-fication Warm Strat. 70°-80° 30-90 Days Cold Strat. 90-180 Days	
Common Persimmon *Diospyros virginiana*	FAC	Tree	T	Oct.-Nov.	Cold Strat. 60-90 Days	

Species	Indicator*	Form	Flood Tolerance**	Seed Dispersal***	Seed Treatments****	Comments
Green Ash *Fraxinus pennsylvanica*	FACW-	Tree	VT	Oct.-Feb.	Cold Strat. 60-90 Days	
Waterlocust *Gleditsia aquatica*	OBL	Tree	T	Sept.-Dec.	Req. Scarifica- tion	
Decidious Holly *Illex deciduas*	FACW	Shrub	VT	Sept.-Mar.	Warm Strat. 68°-Day, 86°-Night 60 Days Cold Strat.-60	
Spicebush *Lindera benzoin*	FACW	Shrub	NE	Sept.-Oct.	Cold Strat. 120 Days	
Sweetgum *Liquidamber styraciflua*	FAC	Tree	T	Sept.-Nov.	Cold Strat. *30* Days	
Sweetbay *Magnolia virginiana*	OBL	Tree	IT	Sept.-Nov.	Cold Strat. 90-180 Days	Can propagate by cuttings
Red Mulberry *Morus rubra*	FACU	Tree	IT	June-Aug.	Cold Strat. 30-90 Days	
Southern Bayberry *Myrica cerifera*	FAC	Shrub	NE	Aug.-Oct.	Cold Strat. 60-90 Days	
Redbay *Persea borbonia*	FACW	Tree	MT	Fall	Not established	

* *Indicator*: **OBL-obligate**; **FACW-facultative wetland**; **FAC-facultative**; **FACU-facultative upland**.
Indicators may be modified by (+) or (-) suffix; (+)indicates a species more frequently found in wetlands; (-) indicates species less frequently found in wetlands.

** *Flood Tolerance Mature Plants*:

VT-Very Tolerant: Survives flooding for periods of two or more growing seasons.
T-Tolerant: Survives flooding for one growing season.
I-Intermediately Tolerant: Survives one to three months of flooding during growing season
WT-Weakly Tolerant: Survives several days to several weeks of growing-season flooding.
IT-Intolerant: Cannot survive even short periods of a few days or weeks of growing-season flooding.
NE-Not established.

*** *Seed Dispersal*: Approximate dates across natural range of a given species.

**** *Seed Treatments*:
Cold stratification: Place moist seeds in polyethylene plastic bags and place in refrigerated storage at 33°-41° F for specified time.
Warm stratification: Place moist seeds in polyethylene plastic bags at 68°-86° F for specified time. Scarification-mechanical or chemical treatment to increase permeability of seed coat.

Table F-8 Seedling response of selected species to flooding conditions

Species	Water Level	Seedling Survival*	Comments
Boxelder *Acer negundo*	Total submersion Growing Season	100% at 2 weeks 70% at 3 weeks 36% at 4 weeks 0% at 32 days	Chlorotic leaves after 4 days. Slow recovery.
Red Maple *Acer rubrum*	Partial submersion Growing season	100% at 5 days 90% at 10 days 0% at 20 days	Adventitious roots developed after 15 days Height growth decreased in saturated soil Soil saturation
	Soil saturation Growing season	Growing season 100% at 32 days	
River Birch *Betula nigra*	Soil saturation Growing season	100% at 32 days	Growth severely stunted
Pecan *Carya illinoensis*	Total submersion Growing season	75% at 4 weeks	
Sugarberry *Celtis laevigata*	Soil saturation Growing season	100% at 60 days	
Common Buttonbush *Cephalanthus occidentalis*	Total submersion Growing season	100% at 30 days	
Green Ash *Fraxinus pennsylvanica*	Total submersion Growing season	100% at 5 days 90% at 10 days 73% at 20 days 20% at 30 days	Lower leaves chlorotic after 8 days Better growth in saturated soil than soil at field capacity
	Partial submersion Growing season	100% at 14 days	
	Soil saturation Growing season	100% at 60 days	
Sweetgum *Liquidambar styraciflua*	Total submersion Growing season	0% at 32 days	
	Partial submersion Growing season	0% at 3 months	
American Sycamore *Platanus occidentalis*	Total submersion Growing season Soil saturation Growing season	100% at 10 days 0% at 30 days 95% at 32 days	Growth decreased by saturated soil
Eastern Cottonwood *Populus deltoids*	Total submersion Growing season	0% at 16 days	Best growth when water table is 2 feet below surface
	Partial submersion Growing season	90% at 10 days 70% at 20 days 47% at 30 days	High mortality when deeply flooded
Cherrybark Oak *Quercus pagoda*	Total submersion Growing season	87% at 5 days 6% at 10 days	Height growth decreased by soil saturation
	Partial submersion Growing season	0% at 20 days 89% at 15 days 47% at 30 days 13% at 60 days	
Water Oak *Quercus nigra*	Partial submersion Growing season	Survived 2 months	
Willow Oak *Quercus phellos*	Soil saturation Growing season	100% at 50 days	Poorer growth in saturated soil than soil at field capacity
Shumard Oak *Quercus shumardii*	Total submersion Growing season	100% at 5 days 90% at 10 days 6% at 20 days	Height growth poorer in saturated soil than soil at field capacity
	Soil saturation Growing season	100% at 30 days 66% at 60 days	

Adapted from Teskey & Hinkley, 1977

* Seeding survival in relation to length of flooding

Table F-8 continued

Species	Water Level	Seedling Survival*	Comments
Black Willow *Salix nigra*	Total submersion Growing season	100% at 30 days	Better height growth in saturated soil than soil at field capacity
	Soil saturation Growing season	100% at 60 days	
Baldcypress *Taxodium distichum* var. *disti-Chum*	Total submersion Growing season	100% at 4 weeks	
American Elm *Ulmus Americana*	Total submersion Growing season	100% at 10 days 27% at 20 days 0% at 30 days	Height growth decreased in saturated soil
	Soil saturation Growing season	100% at 15 days 94% at 60 days	

* Seeding survival in relation to length of flooding

References

Art, Henry W., 1986. A Garden of Wildflowers, 101 Native Species and How to Grow Them, Storey Communications, Inc., Pownal, VT.

Clausen, Ruth Rogers and Ekstrom, Nicolas, H., 1989. Perennials for American Gardens, Random House, New York, NY.

Dirr, Michael A., 1990. Manual of Woody Landscape Plants, Their Identification, Ornamental Characteristics, Culture, Propagation, and Uses, 4th Edition, Stipes Publishing Company, Champaign, IL.

Engineering Technology Associates Inc. and Biohabitats, Inc. (ETA&B), 1993. Design Manual for Use of Bioretention in Stormwater Management, Prince Georges County Dept. of Environmental Resources, Upper Marlboro, MD.

Garber, M.P. and Moorhead, D.J., 1999. Selection, Production and Establishment of Wetland Trees and Shrubs. University of Georgia, College of Agricultural & Environmental Sciences & Daniel B. Warnell School of Forest Resources Cooperative Extension Service.

Georgia Wildlife Web: http://museum.nhm.uga.edu/gawildlife/gawwregions.html

Greenlee, John, (photographed by Derek Fell) 1992. The Encyclopedia of Ornamental Grasses, How to Grow and Use Over 250 Beautiful and Versatile Plants, Rodale Press, Emmas, PA.

Hodler, T.W. and H.A. Schretter. 1986. The Atlas of Georgia. University of Georgia Press, Athens.

Miles, Bebe, 1996. Wildflower Perennials for Your Garden, A Detailed Guide to Years of Bloom from America's Native Heritage, Stackpole Books, Mechanicsburg, PA.

Newcomb, Lawrence, 1977. Newcomb's Wildflower Guide, Little Brown and Company, Boston, MA.

Schueler, Thomas R., July 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMP's, Department of Environmental Programs Metropolitan Washington Council of Governments, Metropolitan Information Center, Washington, DC.

Schueler, Thomas R., October 1996. Design of Stormwater Wetland Systems: Guidelines for Creating Diverse and Effective Stormwater Wetland Systems in the Mid-Atlantic Region, Department of Environmental Programs Metropolitan Washington Council of Governments, Metropolitan Information Center, Washington, D.C.

Schueler, Thomas R. and Claytor, Richard A., 1997. Design of Stormwater Filtering Systems: Appendix B and C, Chesapeake Bay Consortium, Silver Spring, MD.

The Pennsylvania State University, College of Agriculture, Cooperative Extension Service, File No. IVC9 10M386, U. Ed. 85-439 and File No. IVC9 10M587 U.Ed. 86-356, Weed Identification, The Pennsylvania State University, College of Agriculture, Cooperative Extension Service, University Park, PA.

Tiner, Ralph W. Jr., April 1988. Field Guide to Non-Tidal Wetland Identification, U.S. Fish and Wildlife Service, Maryland Department of Natural Resources Maryland Geological

U.S. Army Corps of Engineers, Wetlands Research Program (WRP), 1993. Baseline Site Assessments for Wetland Vegetation Establishment. WRP Technical Note VN-EV-2.1, August 1993.

U.S. Department of Agriculture Hardiness Zone Map. 1999. USDA/ARS Publication Number 1475

Wermund, E.G. 1996. Physiographic Map, The University of Texas at Austin. Bureau of Economic Geology. Austin, Texas

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APPENDIX

G

Storm Water Computer Models and Information Tools

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G.1 Introduction

Storm water management is becoming increasingly complex. The simple notion of collecting runoff and sending it efficiently to the nearest stream is being replaced with considerations of storm water quantity and quality control, infrastructure management, master planning and modeling, financing, complaint tracking, and more. Information needs are critical to a successful local program. North Central Texas communities need to both invest in and be aware of new and emerging technologies that can provide the ability to collect, organize, maintain and effectively use vast amounts of data and information for their community's storm water management activities.

There is a great deal of computer software that has been developed based on the intensive research effort in urban hydrology, hydraulics, and storm water quality. Computer models use the computational power of computers to automate the tedious and time-consuming manual calculations. Most models also include extensive routines for data management, including input and output procedures, and possibly including graphics and statistical capabilities.

Computer modeling became an integral part of storm drainage planning and design in the mid-1970s. Several agencies undertook major software developments and these were soon supplemented by a plethora of proprietary models, many of which were simply variants on the originals. The proliferation of personal computers in the 1990s has made it possible for virtually every engineer to use state-of-the-art analytical technology for purposes ranging from analysis of individual pipes to comprehensive storm water management plans for entire cities.

In addition to the simulation of hydrologic and hydraulic processes, computer models can have other uses. They can provide a quantitative means to test alternatives and controls before implementation of expensive measures in the field. If a model has been calibrated and verified at a minimum of one site, it may be used to simulate non-monitored conditions and to extrapolate results to similar ungauged sites. Models may be used to extend time series of flows, stages and quality parameters beyond the duration of measurements, from which statistical performance measures then may be derived. They may also be used for design optimization and real-time control.

A local staff or design engineer will typically use one or more of these pieces of software in storm water facility design and review, according to the design objectives and available resources. However, it should be kept in mind that proper use of computer modeling packages requires a good knowledge of the operations of the software model and any assumptions that the model makes. The engineer should have knowledge of the hydrological, hydraulic and water quality processes simulated and knowledge of the algorithms employed by the model to perform the simulation.

G.2 Types of Models

In urban storm water management there are typically three types of computer models that are commonly used: *hydrologic*, *hydraulic* and *water quality* models. There are also a number of other specialty models to simulate ancillary issues (some of which are sub-sets of the three main categories) such as sediment transport, channel stability, lake quality, dissolved oxygen, evapotranspiration, etc.

G.2.1 Hydrologic Models

Hydrologic models attempt to simulate the rainfall-runoff process to tell us “how much water, how often.” They use rainfall information and watershed characteristics to provide runoff data including peak flow, flood hydrograph, and flow frequencies. Hydrologic models can be either:

- *Deterministic* – giving one answer for a specific input set, or
- *Stochastic* – involving random inputs giving any number of responses for a given set of parameters;

- *Continuous* – simulating many storm events over a period of time, or
- *Single Event* – simulating one storm event;

- *Lumped* – representing a large area of land use by a single set of parameters, or
- *Distributed* – land areas are broken into many small homogeneous areas each of which has a complete hydrologic calculation made on it.

G.2.2 Hydraulic Models

Hydraulic models take a known flow amount (typically the output of a hydrologic model) and provide information about flow height, location, velocity, direction, and pressure. Hydraulic models share some of the differing characteristics of hydrologic models (continuous vs. single event) and add the following:

- *One-dimensional* – calculating flow information in one direction (e.g. downstream) only, or
- *Multi-dimensional* – calculating flow information in several dimensions (e.g. in and out of the channel and downstream);

- *Steady* – having a single unchanging flow velocity value at a point in the system, or
- *Unsteady* – having changing flow velocities with time;

- *Uniform* – assuming the channel slope and energy slope are equal, or
- *Non-uniform* – solving a more complex formulation of the energy and momentum equations to account for the dynamic nature of flows.

For most problems encountered in hydraulics, a simple one-dimensional, steady model will work well. But if the volume and time distribution of flow are important (for example, in a steeper stream with storage behind a series of high culvert embankments) an unsteady model is needed. If there is a need to predict with accuracy the ebb and flow of floodwater out of a channel (for example in a wide, flat floodplain where there are relief openings under a road) then a 2-dimensional model becomes necessary. If pressure flow and the accurate computation of a hydraulic grade line are important an unsteady, non-uniform model with pressure flow calculating capabilities is needed.

G.2.3 Water Quality Models

The goal in water quality modeling is to adequately simulate the various processes and interactions of storm water pollution. Water quality models have been developed with an ability to predict loadings of various types of storm water pollutants.

Water quality models can become very complex if the complete cycle of buildup, wash-off, and impact are determined. These models share the various features of hydrologic and hydraulic models in that it is the runoff flow that carries the pollutants. Therefore, a continuous hydrologic model with estimated pollution concentrations becomes a continuous water quality pollution model. Water quality models can reflect pollution from both point and nonpoint sources.

Water quality models tend to have applications that are targeted toward specific pollutants, source types, or receiving waters. Some models involve biological processes as well as physical and chemical processes. Often great simplifications or gross assumptions are necessary to be able to model pollutant accumulations, transformations, and eventual impacts.

Detailed short time increment predictions of “pollutographs” are seldom needed for the assessment of receiving water quality. Hence, the total storm event loads or mean concentrations are normally adequate. Simple spreadsheet-based loading models involve an estimate of the runoff volume which, when multiplied by an event mean concentration, provide an estimate of pollution loading. Because of the lack of ability to calibrate such models for variable physical parameters, such simple models tend to be more accurate the longer the time period over which the pollution load is averaged. An annual pollutant load prediction may tend toward a central estimate, while any specific storm prediction may be

grossly in error when compared to actual loadings because antecedent conditions vary widely from week to week. Simulation models have the ability to adjust a number of loading parameters for calibration purposes and can simulate pollution accumulation over a long period. They can then more reliably predict loadings for any specific storm event.

While calibration data is not always needed in hydrologic or hydraulic models for an acceptably accurate answer, in water quality models the non-calibrated prediction is often off by orders of magnitude. Water quality predictions are not credible without adequate site-specific data for calibration and verification. However, even without specifically accurate loading values, relative effects of pollution abatement controls can be tested using uncalibrated models.

G.2.4 Computer Model Applications

Storm water computer models can also be categorized by their use or application:

Screening-level models are typically equations or spreadsheet models that give a first estimate of the magnitude of urban runoff quality or quantity. At times this is the only level that is necessary to provide answers. This is true either because the answer needs to be only approximate or because there is no data to justify a more refined procedure.

Planning-level models are used to perform “what if” analysis comparing, in a general way, design alternatives or control options. They are used to establish flow frequencies, floodplain boundaries, and general pollution loading values.

Design-level models are oriented toward the detailed simulation of a single storm event for the purposes of urban storm water design. They provide a more complete description of flow or pollution values anywhere in the system of concern and allow for adjustment of various input and output variables in some detail. They can be more exact in the impact of control options, and tend to have a better ability to be calibrated to fit observed data.

Operational models are used to produce actual control decisions during a storm event. They are often linked with SCADA systems. They are often developed from modified or strongly calibrated design models, or can be developed on a site-specific basis to appropriately link with the system of concern and accurately model the important physical phenomena.

G.3 The Modeling Process

G.3.1 Basic Computer Modeling Principles

The following basic principles apply to all forms of computer modeling:

1. All computer models require site-specific information to be supplied by the user. Inputs are the measured or estimated parameters the model needs to make calculations. For example, basic hydrologic models might include: area, slope, land use, channel forms and roughness, connectivity, and rainfall. A basic hydraulic model would include: channel slope, discharge, roughness, shape, obstructions or constrictions, and connectivity. Water quality models may add pollution loading or build-up-washoff factors, and fate and transport information. All models, for planning and design, allow the modeler to try different combinations of variables to see what happens (called a “what if" analysis).
2. While modeling generally yields more information, simpler methods may provide sufficient information for design or solving management issues. In general, the simplest method that provides the desired analysis should be used. The risk of using a more complex (and presumably "better") model is that it requires more expertise, data, support, and resources to use and understand. There is also a consequent higher probability of misapplication.

3. If water quality problems are being considered, it still may not be necessary to simulate quality processes since most control strategies are based on hydrologic or hydraulic considerations. Quality processes are very difficult to simulate accurately. If abatement strategies can be developed without the simulation of water quality parameters, the overall modeling program will be greatly simplified.

Models sometimes may be used to extrapolate beyond the record of measured data record. It is important to recognize, however, that models do not extend data, but rather generate simulated numbers that should never be assumed to be the same as data collected in the field. Careful consideration should be given when using models to provide input to receiving water quality analyses. The quality response of most receiving waters is relatively insensitive to such short-term variations. In many instances, the total storm load will suffice to determine the receiving water response. Simulation of short time increment changes in concentrations and loads is generally necessary only for analysis of control options, such as storage or high-rate treatment, whose efficiency may depend on the transient behavior of the quality constituents.

The overall modeling process involves: (1) development of study or model objectives, (2) identification of resources and constraints, and finally, (3) the selection and implementation of the model itself.

G.3.2 Model Objectives

It is important to know specifically to what accuracy and in what format the answers are needed. Requiring a simple peak flow is far different from needing to know the timing of peaks from several different intersecting watersheds. Estimating future floodplain elevations along a reach is a fundamentally different problem than finding the probability of roadway overtopping.

A review of the problem begins the process of determining the model objectives. These objectives also establish a performance or design criteria for the model. Must the system handle the 25-year storm? Are future conditions important? Which ones? Are annual loadings of pollution adequate? Which pollutants?

Those aspects of the system to be modeled will dictate what models are appropriate for use. For example, if storm sewers are present, then an open channel model can be ruled out as an appropriate model for the entire system. If a specific type of hydraulic structure is present that a standard model cannot handle, an alternate way to simulate that structure will be necessary.

Model objectives also explain how the numbers generated from the model will relate to the needs of the study. For example, if a cost-benefit analysis is required, the model results must be interpreted in terms of overall life-cycle cost and not simply in terms of discharge rate.

G.3.3 Model Constraints

Availability of data, funds, time, and user ability can potentially constrain modeling solutions. The goal of any modeling effort is to develop an approach that stays within the constraints dictated while addressing the needs of the study identified in the previous step. Data collection/availability and cost are usually the chief constraints.

Sources of existing available data should be researched. Look for data that tends to “ground truth” model outputs. Even partial data can be useful if it helps to validate the model or modeling results. After existing data sources have been identified, the need to gather additional data is assessed. Automated processes and systems such as GIS and GPS can reduce both cost and human error. A consideration of the long-term use of data and its maintenance is necessary. For example, if the model is to eventually become an operational model, the ability to maintain the data in a cost effective way becomes of paramount importance.

Accuracy and the corresponding necessary level of detail are of overriding importance. Accuracy depends on both the accuracy of the input data and the degree to which the model adequately represents the hydrologic, hydraulic, or water quality processes being modeled. For example, if lumped hydrologic

parameters are adequate, then the cost of the modeling effort can be reduced. However, the ability to determine information within the sub-basin represented by a single parameter is lost. Changing model needs from an average 500-acre sub-basin size to a 50-acre size can increase the cost of a model almost ten fold. Is the information derived worth the cost?

Both risk and uncertainty affect the modeler's ability to predict results accurately. Risk is an estimated chance of an occurrence, such as flooding. Uncertainty is the error associated with measuring or estimating key parameters or functions. Uncertainty arises due to errors in sampling, measurement, estimation and forecasting, and modeling. For hydrologic and hydraulic analysis, stage and discharge are of prime importance. Uncertainty in discharge is due to short or nonexistent flood records, inaccurate rainfall-runoff modeling, and inaccuracy in known flood flow regulation where it exists. Stage uncertainty comes from errors and unknowns in roughness, geometry, debris accumulation, sediment effects, and other factors.

Accuracy developed in one area can be impacted by rough estimates in another, and the technological gains lost. For example, the gains in accuracy from very precise field surveys of cross sections can be lost if the estimates of roughness coefficients or discharge rates are very approximate.

Sensitivity analysis involves holding all parameters constant except one and assessing the change in the output variable of concern with a certain percent change in the input variable. Those variables that are amplified in the output should be estimated with higher accuracy and with a more detailed consideration of the potential range of values and the need for conservative design. The modeler must try to assess how accurate estimates are and to account for risk and uncertainty through estimating the range of potential error and choosing values that balance conservative engineering with cost consciousness. The designer typically develops a "most likely" estimate of a certain design parameter (for example, 10-year storm rainfall or Manning's roughness coefficient) and then uses sensitivity analysis to test the impact of variability in the parameter estimate on the final solution

G.3.4 Selection and Implementation

Once the model objectives and constraints have been evaluated, the model (or models) is selected and the study or design is implemented. Typical steps in model implementation include *validation*, *calibration*, *verification,* and *production*.

Validation involves a determination that the model is structured and coded as intended for the range of variables to be encountered in the study. Validation tests key algorithms for accuracy. For example, if a hydrologic model cannot handle short time steps or long time periods, it cannot be used without modification. If a certain model begins to lose accuracy at high or low imperviousness or cannot accurately handle backwater situations, and these will be encountered in practice the model cannot be used. Often validation is a one-time effort, after which the modeler is comfortable with the model's "quirks" and knows how to deal with them. Validation often involves pushing parameters to the limit of reasonable extent to test an algorithm. For example, in a hydrologic model infiltration can be reduced to zero to test if the input and output hydrographs are equal. Or the model can be run with small rainfalls using porous soils to determine if no runoff is generated, or only runoff from directly connected impervious areas.

Calibration is the comparison of a model to field measurements, other known estimates of output (e.g. regression equations), or another model known to be accurate, and the subsequent adjustment of the model to best fit those measurements.

Verification then tests the calibrated model against another set of data not used in the calibration. This step is not always possible due to the general shortage of other data. Goodness of prediction is done through a simple comparison of the difference in observed and predicted peaks, pollution loads, flood elevations or volumes divided by the observed values and expressed as a percentage, or as simple ratio. Assessing the goodness of fit of a hydrograph is done by calculating the sum of the squares of the difference between observed and predicted values at discreet time steps.

Once the model is prepared for use, attention shifts to efficient *production* methods that minimize the potential for errors while maximizing efficiency. Often "production line" efforts are used for large modeling projects. However, constant attention must be paid to ensure the execution of correct procedures, detailed documentation of efforts and input/output data sets, and recognition of anomalies that would invalidate a particular model run.

While there is much to be gained from simple user interfaces and black box approaches that simplify the input and output processes, there is an inherent danger that the modeler will not be aware of errors or problems in the modeling process. For example, in hydraulic modeling, shifts from super- to sub-critical flow happen at sharp break points and are reflected in a jump in water surface. If not caught, a model will under predict flow elevation. Numeric instability in mathematical algorithms may give oscillating answers that have nothing to do with reality. A structured review process must be established to insure reasonableness of output and accuracy of input values has been used. Labeling of data sets should be systematic and exact.

G.4 Summary of Commonly-Used Models

Computer models can be simple, representing only a very few measured or estimated input parameters or can be very complex involving twenty times the number of input parameters. The "right" model is the one that: (1) the user thoroughly understands, (2) gives adequately accurate and clearly displayed answers to the key questions, (3) minimizes time and cost, and (4) uses readily available or collected information. Complex models used to answer simple questions are not an advantage. However, simple models that do not model key necessary physical processes are useless.

There is no one engineering model or software that addresses all hydrologic, hydraulic, and water quality situations. Design needs and troubleshooting for watershed and storm water management occur on several different scales and can be either system-wide (i.e., watershed) or localized. System-wide issues can occur on both large and small drainage systems, but generally require detailed, and often expensive, watershed models and/or design tools. The program(s) chosen to address these issues should handle both major and minor drainage systems. Localized issues also exist on both major and minor drainage systems, but unlike system-wide problems, flood and water quality solution alternatives can usually be developed quickly and cheaply using simpler engineering methods and design tools.

Table G.4-1 lists several widely used computer programs and modeling packages. The programs were examined for their applicability to both system-wide and localized issues, the methodologies used for computations, and ease-of-use.

For the purposes of this table, major drainage systems are defined as those receiving flow from larger watersheds. These are typically FEMA-regulated streams, lakes, or reservoirs. Minor drainage systems are smaller natural and man-made systems that drain to the more major streams. Minor drainage systems can have both closed and open-channel components and can include, but are not limited to, neighborhood storm sewers, culverts, ditches, and tributaries.

A brief description of program capabilities and methodologies are presented in a short discussion of each program.

Table G.4-1 Storm Water Modeling Programs and Design Tools

	Major System Modeling	Minor System Modeling	Hydrologic Features	Hydraulic Features	Water Quality Features
Hydrology Software					
HEC-HMS (HEC-1)	X		X		
TR-55			X		
PondPack		X	X	X	
WMS	X		X		
Watershed Modeling	X		X		
Hydraulics Software					
HEC-RAS (HEC-2)	X			X	
WSPRO	X			X	
EPA SWMM	X	X	X	X	X
CulvertMaster		X		X	
FlowMaster		X		X	
Water Quality Software					
HSPF	X		X		X
BASINS	X		X	X	X
QUAL2E	X			X	X
WASP5	X			X	X
SLAMM	X		X		X

G.4.1 Hydrology Programs

HEC-HMS (HEC-1) - Flood Hydrograph Package

HEC-1 was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers to simulate the surface runoff response of a watershed to rainfall events. Although it is a DOS-based program, it is still considered by many in the engineering and regulatory communities to be the leading model for major drainage system applications such as Flood Insurance Studies and watershed master planning. The Federal Emergency Management Agency (FEMA) accepts both HEC-1 and HEC-HMS.

In a HEC-HMS model, the watershed is represented in the model as an interconnected system of hydrologic (e.g., sub-basins, reservoirs, ponds) and hydraulic (e.g., channels, closed conduits, pumps) components. The model computes a runoff hydrograph at each component, combining two or more hydrographs as it moves downstream in the watershed. The model has a variety of rainfall-runoff simulation methods, including the popular SCS Curve Number methodology. The user can define rainfall events using gage or historical data, or HEC-HMS can generate synthetic storms. Hydrograph generation is performed using the unit hydrograph technique. Clark, SCS Dimensionless, and Snyder Unit Hydrographs are the available methodologies. Several common channel and storage routing techniques are available as well.

HEC-HMS is not considered a "design tool." The program has limited hydraulic capabilities. It does not account for tailwater effects and cannot adequately simulate many urban hydraulic structures such as pipe networks, culverts, and multi-stage detention pond outlet structures. However, there are other hydrologic applications developed within HEC-HMS that have been utilized with much success. Multiplan-multiflood analyses allow the user to simulate a number of flood events for different watershed situations (or plans). The dam safety option enables the user to analyze the impact dam overtopping or structural failure on downstream areas. Flood damage analyses assess the economic impact of flood damage.

Because it is not a Windows-based program, HEC-1 does not have easy-to-use input and output report generation and graphical capabilities, and therefore is generally not considered a user-friendly program. Because of its wide acceptance, several software development companies have incorporated the source code into enhanced "shells" to provide a user-friendly interface and graphical input and output capabilities. Examples of these programs include Graphical HEC-1 developed by Haested Methods and WMS developed by the Environmental Modeling Research Laboratory.

The Corps of Engineers developed a user-friendly, Windows-based Hydrologic Modeling System (HEC-HMS) intended to replace the DOS-based HEC-1 model. HEC-HMS has all the components of HEC-1, with more user-friendly input and output processors and graphical capabilities. HEC-1 files can be imported into HEC-HMS.

TR-55 - Technical Release 55

The TR-55 model is a DOS-based software package used for estimating runoff hydrographs and peak discharges for small urban watersheds. The model was developed by the NRCS (formally SCS), and therefore uses SCS hydrograph methodology to estimate runoff. No other methodology is available in the program. Four 24-hour regional rainfall distributions are available for use. Rainfall durations less than 24-hours cannot be simulated. Using detailed input data entered by the user, the TR-55 model can calculate the area-weighted CN, time of concentration and travel time. Detention pond (i.e., storage) analysis is also available in the TR-55 model, and is intended for initial pond sizing. Final design requires a more detailed analysis.

A Windows-based version (Win TR-55) has been developed by the NRCS. Haestad Methods, Inc., included most of the TR-55 capabilities in its PondPack program described below. Both Win TR-55 and Pond Pack are models acceptable to FEMA.

PONDPACK

PondPack, by Haestad Methods, Inc., is Windows-based software developed for modeling general hydrology and runoff from site development. The program analyzes pre- and post-developed watershed conditions and sizes detention ponds. It also computes outlet rating-curves with consideration of tailwater effects, accounts for pond infiltration, calculates detention times and analyzes channels.

Rainfall options are unlimited. The user can model any duration or distribution, for synthetic or real storm events. Several peak discharge and hydrograph computation methods are available, including SCS, the Rational Method and the Santa Barbara Unit Hydrograph procedure. Infiltration can be considered, and pond and channel routing options are available as well. Like TR-55, PondPack allows the user to calculate hydrologic parameters, such as the time of concentration, within the program.

PondPack has limited, but useful hydraulic features, using Manning's equation to model natural and man-made channels and pipes. A wide variety of detention pond outlet structure configurations can be modeled, including low flow culverts, weirs, riser pipes, and even user-defined structures.

WMS - Watershed Modeling System

WMS was developed by the Engineer Computer Graphics Laboratory of Brigham Young University. WMS is a Windows-based user interface that provides a link between terrain models and GIS software, with industry standard lumped parameter hydrologic models, including HEC-1, TR-55, TR-20 and others. The hydrologic models can be run from the WMS interface. The link between the spatial terrain data and the hydrologic model(s) gives the user the ability to develop hydrologic data that is typically gathered using manual methods from within the program. For example, when using SCS methodologies, the user can delineate watersheds and sub-basins, determine areas and curve numbers, and calculate the time of concentration at the computer. Typically, these computations are done manually, and are laborious and time-consuming. WMS attempts to utilize digital spatial data to make these tasks more efficient.

Watershed Modeling

The Watershed Modeling program was developed to compute runoff and design flood control structures. The program can run inside the MicroStation CAD system. Like WMS, this feature enables the program

to delineate and analyze the drainage area of interest. Area, curve number, land use and other hydrologic parameters can be computed and/or catalogued for the user, removing much of the manual calculation typically performed by the hydrologic modeler.

Watershed Modeling contains a variety of methods to calculate flood hydrographs, including SCS, Snyder and Rational methods. Rainfall can be synthetic or user-defined, with any duration and return period. Rainfall maps for the entire U.S. are provided to help the user calculate IDF relationships. Several techniques are available for channel and storage routing. The user also has a wide variety of outlet structure options for detention pond analysis and design.

G.4.2 Hydraulics Programs

HEC-RAS - River Analysis System

HEC-RAS is a Windows-based hydraulic model developed by the Corps of Engineers to replace the popular, DOS-based HEC-2 model. RAS has the ability to import and convert HEC-2 input files and expands upon the capabilities of HEC-2. Since its introduction several years ago, the user-friendly HEC-RAS has become known as an excellent model for simulation of major systems (i.e., open channel flow) and has become the chief model for calculating floodplain elevations and determining floodway encroachments for Flood Insurance Studies. Like HEC-2, HEC-RAS has been accepted for FIS analysis by the FEMA. However, RAS is a much easier model to use than HEC-2 as it has an extremely useful interface that provides the immediate capability to view model input and output data in graphical, tabular, and report formats.

HEC-RAS performs one-dimensional analysis for steady flow water surface profiles, using the energy equation. Energy losses are calculated using Manning's equation and contraction and expansion changes. Rapidly varied flow (e.g., hydraulic jumps) is modeled using the momentum equation. The effects of in-stream structures, such as bridges, culverts, weirs, floodplain obstructions, and in-stream changes such as levees and channel improvements can be simulated. The model allows the user to define the geometry of the channel or structure to the level of detail required by the application. One popular and useful feature of the HEC-RAS model is the capability to easily facilitate floodway encroachment analysis. Five encroachment methods are available to the user.

The HEC-RAS model has components for unsteady flow and has sediment transport simulations under development. HEC-GeoRAS provides an interface with GIS platforms which allows the preparation of input data for import into HEC-RAS. Several software developers have released other enhanced versions of RAS that provide the capability to import GIS data for channel geometry and export RAS output for floodplain and floodway delineation. Examples of such software include BOSS RMS, developed by BOSS International and SMS (Surface Water Modeling System), distributed by the Scientific Software Group.

WSPRO

WSPRO was developed by the USGS to compute water surface profiles for one-dimensional, gradually varied, steady flow. Like HEC-RAS, WSPRO can develop profiles in subcritical, critical, and supercritical flow regimes. WSPRO is designated HY-7 in the Federal Highway Administration ICPR (Interconnected Pond Routing) Model, and its original objective was analysis and design of bridge openings and embankment configurations. Since then, the model has been expanded to model open channels and culverts.

Open channel computations use standard step-backwater techniques. Flow through bridges is simulated using an energy-balancing technique that uses a coefficient of discharge and estimates an effective flow length. Pressure flow under bridges is simulated using orifice-type flow equations developed by the FHWA. Culvert flow is simulated using FHWA techniques for inlet control and energy balance for outlet control.

WSPRO is considered a fairly easy-to-use DOS-based model, applicable to water surface profile analysis for highway design, flood insurance studies, and establishing stage-discharge relationships. However,

the model in its original form is not Windows based and therefore does not have the useful editing and graphical features found in HEC-RAS. Like HEC-RAS, a third party software developer has designed SMS (Surface Water Modeling Software) to support both pre- and post-processing of WSPRO data.

EPA SWMM - Storm Water Management Model

EPA SWMM was developed by the Environmental Protection Agency (EPA) to analyze storm water quantity and quality problems associated with runoff from urban areas. EPA SWMM has become the model of choice for simulation of minor drainage systems primarily composed of closed conduits. The model can simulate both single-event and continuous events and has the capability to model both wet and dry weather flow. The basic output from SWMM consists of runoff hydrographs, pollutographs, storage volumes, and flow stages and depths.

SWMM's hydraulic computations are link-node based and are performed in separate modules, called blocks. The EXTRAN computational block solves complete dynamic flow routing equations to simulate backwater, looped pipe connections, manhole surcharging, and pressure flow. It is the most comprehensive model in its capabilities to simulate urban storm flow and many cities have used it successfully for storm water, sanitary, or combined sewer system modeling. Open channel flow can be simulated using the TRANSPORT block, which solves the kinematic wave equations for natural channel cross-sections.

Although evaluated for this study as a hydraulic model, SWMM has both hydrologic and water quality components. Hydrologic processes are simulated using the RUNOFF block, which computes the quantity and quality of runoff from drainage areas and routes the flow to the major sewer system lines. Pollutant transport is simulated in tandem with hydrologic and hydraulic computations and consists of calculation of pollutant buildup and washoff from land surfaces and pollutant routing, scour, and in-conduit suspension in flow conduits and channels.

EPA SWMM is a public domain, DOS-based model. For large watersheds with extensive pipe networks, input and output processing can be tedious and confusing. Because of the popularity of the model commercial, third-party enhancements to SWMM have become more common, making the model a strong choice for minor system drainage modeling. Examples of commercially enhanced versions of EPA SWMM include MIKE SWMM, distributed by BOSS International, XPSWMM by XP-Software, and PCSWMM by Computational Hydraulics Inc (CHI). CHI also developed PCSWMM GIS, which ties the SWMM model to a GIS platform.

CULVERTMASTER

CulvertMaster, developed by Haestad Methods, Inc., is an easy-to-use, Windows-based culvert simulation and design program. The program can analyze pressure or free surface flow conditions. CulvertMaster can handle drawdown and backwater conditions, including composite profiles and hydraulic jumps. A variety of common culvert shapes and section types are available. Tailwater effects are considered and user can enter a constant tailwater elevation, a rating curve, or specify an outlet channel section. Culvert hydraulics are solved using FHWA methodology for inlet and outlet control computations. Roadway and weir overtopping are checked in the solution of the culvert.

CulvertMaster does have a hydrologic analysis component to determine peak flow using the Rational Method, SCS Graphical Peak Methods. The user also has the option of entering a known peak flow rate. The user must enter all rainfall and runoff information (e.g., IDF data, rainfall depths, curve numbers, C coefficients, etc).

FLOWMASTER

FlowMaster, also developed by Haestad Methods, Inc., is a Windows-based hydraulic pipe and channel design program. The user enters known information on the channel section or pipe, and allows the program to solve for the unknown parameter(s), such as diameter, depth, slope, roughness, capacity, velocity, etc. Solution methods include Manning's equation, the Darcy-Weisbach formula, Hazen-Williams formula, and Kutter's Formula. The program also features calculations for weirs, orifices, gutter flow, ditch, and median flow and discharge into curb, grated, and slot inlets.

Other Hydraulics Programs
In addition to the Hydraulics Software listed in Table G.4-1, appropriately applied versions of the following computer models are also available for use.

- *CULVERT* – by TxDOT for hydraulic analysis and design of culverts.
- *WINSTORM* – by TxDOT for analysis and design of storm sewer.
- *HY8* – by the Federal Highway Association (FHWA) for hydraulic analysis and design of culverts.
- *HY8Energy* – by FHWA for the selection and analysis of energy dissipation structures.
- *CHANLPRO* – by U.S. Army Corps of Engineers for analysis and design of streambank protection.

G.4.3 Water Quality Programs

HSPF - Hydrologic Simulation Program FORTRAN
The HSPF model was developed by the EPA for the continuous or single-event simulation of runoff quantity and quality from a watershed. The original model was developed from the Stanford Watershed Model, which simulated runoff quantity only. It was expanded to include quality components, and has since become a popular model for continuous nonpoint source water quality simulations. Nonpoint source conventional and toxic organic pollutants from urban and agricultural land uses can be simulated on pervious and impervious land surfaces and in streams and well-mixed impoundments. The various hydrologic processes are represented mathematically as flows and storages. The watershed is divided into land segments, channel reaches, and reservoirs. Water, sediment, and pollutants leaving a land segment move laterally to a downstream land segment, a stream or river reach, or reservoir. Infiltration is considered for pervious land segments.

HSPF model output includes time series information for water quality and quantity, flow rates, sediment loads, and nutrient and pesticide concentrations. To manage the large amounts of data associated with the model, HSPF includes a database management system. To date, HSPF is still a DOS-based model and therefore does not have the useful graphical and editing options of a Windows-based program. Input data requirements for the model are extensive and the model takes some time to learn. However the EPA continues to expand and develop HSPF, and still recommends it for the continuous simulation of hydrology and water quality in watersheds.

BASINS - Better Assessment Science Integrating Point and NonPoint Sources
The BASINS watershed analysis system was developed by the EPA for use by regional, state, and local pollution control agencies to analyze water quality on a watershed-wide basis. BASINS integrates the ArcView GIS environment, national databases containing watershed data, modeling programs, and water quality assessment tools into one stand-alone program. The program will analyze both point and nonpoint sources and supports the development of the total maximum daily loads (TMDLs). The assessment tools and models utilized in BASINS include TARGET, ASSESS, Data Mining, HSPF, TOXIROUTE, and QUAL2E. The databases, assessment tools and models are directly tied to the ArcView GIS environment.

QUAL2EU - Enhanced Stream Water Quality Model
QUAL2EU was developed by the EPA and intended for use as a water quality planning tool. The model actually consists of four modules: QUAL2E - the original water quality model, QUAL2EU - the water quality model with uncertainty analysis, and pre- and post-processing modules. QUAL2EU simulates steady state or dynamic conditions in branching streams and well-mixed lakes, and can evaluate the impact of waste loads on water quality. It also can enhance a field sampling program by helping to identify the magnitude and quality characteristics of nonpoint waste loads. Up to 15 water quality constituents can be modeled. Dynamic simulation allows the user to study the effects of diurnal variations in water quality (primarily DO and temperature). The steady state option allows the user to perform uncertainty analyses.

QUAL2EU is a DOS-based program, and the user will require some length of time to develop a QUAL2EU model, mainly due to the complexity of the model and data requirements for a simulation.

However, to ease user interaction with the model an interactive preprocessor (AQUAL2) has been developed to help the user build input data files. A postprocessor (Q2PLOT) also exists that displays model output in textual or graphical formats.

WASP5 - Water Quality Analysis Simulation Program
The WASP5 model was developed by the EPA to simulate contaminant fate in surface waters. Both chemical and toxic pollution can be simulated in one, two, or three dimensions. Problems studied using WASP5 include biochemical oxygen demand and dissolved oxygen dynamics, nutrients and eutrophication, bacterial contamination, and organic chemical and heavy metal contamination. WASP5 has an associated stand alone hydrodynamic model called DYNHYD5 that simulates variable tidal cycles, wind, and unsteady flows. DYNHYD4 supplies flows and volumes to the water quality model. The model is DOS-based; however WASP packages can be obtained from outside vendors that include interactive tabular and graphical pre- and post-processors.

SLAMM - Source Loading and Management Model
The SLAMM model was originally developed as a planning tool to model runoff water quality changes resulting from urban runoff pollutants. The model has been expanded to included simulation of common water quality best management practices such as infiltration BMPs, wet detention ponds, porous pavement, street cleaning, catchbasin cleaning, and grass swales. Unlike other water quality models, SLAMM focuses on small storm hydrology and pollutant washoff which is large contributor to urban stream water quality problems. SLAMM computations are based on field observations, as opposed to theoretical processes. SLAMM can be used in conjunction with more commonly used hydrologic models to predict pollutant sources and flows.

G.5 Information Tools

G.5.1 Geographic Information Systems (GIS)

A Geographic Information System (GIS) is a computer-based database system designed to spatially analyze and display data. A GIS stores information about a given area as a collection of thematic layers that can be linked together by geo-referencing. This simple but extremely powerful and versatile concept has proven invaluable for solving many real-world storm water problems from tracking complaints to master planning applications and infrastructure management.

GIS Components

A functional GIS integrates four key components: hardware, software, data, and trained users:

- *Hardware* – Desktop computers and digitizing equipment are the primary hardware components of a typical local GIS system.
- *Software* – GIS software provides the functionality and tools needed to capture or input, store, analyze, display, and output geographic information.
- *Data* – Generally, the most costly part of a GIS is data development. Some geographic data and related tabular data can be collected in-house or purchased from commercial data providers. A GIS can also integrate tabular data or electronic drafting (CAD) data to build information into the GIS database.
- *Users* – GIS technology is of limited value without trained operators who understand the data, system, organization and how to apply the resources to achieve the desired results.

GIS Functions

General purpose geographic information systems essentially perform four processes or tasks:

1. Data Input – Before geographic data can be used in a GIS, the data must be converted into a suitable digital format. The process of converting data from paper maps into computer files is called digitizing. Modern GIS technology can sometimes automate this process fully for projects using scanning technology. Some jobs may require some manual digitizing using a digitizing table. Today many types of geographic data already exist in GIS-compatible formats. These data can be obtained from a number of different sources including the North Central Texas GIS Data Clearinghouse at *http://gis.dfwinfo.com/geodata/.*

2. Data Conversion – It is likely that some needed data may not be in the correct format or proper map projection to use with your system. Most GIS software has the ability to do this conversion, but in some cases this is better done by a contractor who specializes in data conversion. Be careful with third party data. It is imperative that you understand the source, quality, age, accuracy, and limitations of a dataset. This and other information about a dataset is often provided in (FGDG) metadata that accompanies the dataset.

3. Query and Analysis – Once there is a functioning GIS database containing geographic information, it can be queried to answer questions such as:

- Who owns the land parcel being flooded?
- What is the distance between two stream locations?
- Which homes are located in the updated floodplain?
- How will the new development impact downstream properties?
- What types of infrastructure give us the most complaints and where are they located?

GIS provides both simple point-and-click query capabilities and sophisticated spatial analysis tools to provide timely information to storm water managers and analysts alike. GIS technology can also be used to analyze geographic data to look for patterns and trends and to undertake "what if" scenarios. Most modern GIS's have many powerful analytical tools including:

- *Size Analysis* – Provides specific information about a feature. (e.g. What is the area and perimeter of a parcel?)
- *Proximity Analysis* – Determines relationships between objects and areas. (e.g. Who is located within 100 feet of the streambank?)
- *Overlay Analysis* – Performs integration of different data layers. (e.g. What is the SCS curve number for this sub-watershed considering soils and land use?) Figure G.3-1 illustrates the overlay concept.
- *Network Analysis* – Analyzes the connectivity of linear features and establishes routes or direction of flow. (e.g. Which pipes feed into this junction box?)
- *Raster Analysis* – Utilizes a raster model to address a number of hydrologic issues. (e.g. What does the 3-D model of this watershed look like? Where does the water flow?)

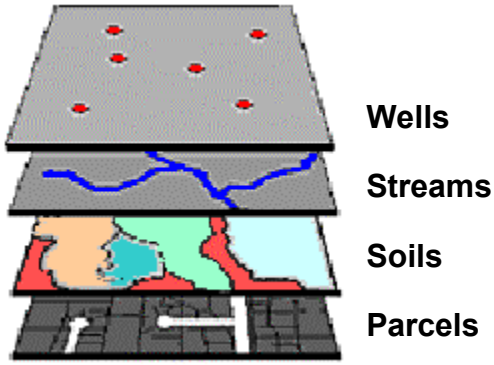

Figure G.5-1 Example of Overlay Analysis
Source: ESRI

4. Data Display, Output, and Visualization – Geographic information systems excel at being able to create rich and detailed maps, graphs, and other types of output which allow local staff, elected officials, and the general public to be able to visualize and understand complex problems and large amounts of information. These maps and charts can be integrated with reports, three-dimensional views, photographic images, and multimedia presentations.

Use of GIS in Storm Water Management

GIS can be useful to a community in a wide variety of storm water-related applications:

- GIS can be used for the mapping of surface features, land uses, soils, rainfall amounts, watershed boundaries, slopes, land cover, etc.
- A GIS can manage a storm water system inventory and information about facility conditions, storm sewer networks, maintenance scheduling, and problem areas.
- GIS can be used to automate certain tasks such as measuring the areas of subwatersheds, plotting floodplain boundaries, or assessing storm water utility fees. Figure G.5-2 shows an example of automated hydrologic mapping.
- A GIS can be used to evaluate water quality impacts and answer cause and effect questions, such as the relationship between various land uses and in-stream pollution monitoring results.
- "What if" analyses can be undertaken with GIS. For example, various land use scenarios and their impacts on pollution or flooding can be tried in various combinations to determine the best management solutions or to determine the outcome of current decisions. When tied to hydrology, hydraulics, and/or water quality models, this type of analysis becomes a powerful tool to assess the impacts of new development on downstream properties. For example, Figure G.5-3 shows the flooding impacts on a small tributary for a proposed new development approved during a rezoning.
- GIS databases can provide staff, elected officials, and citizens with immediate answers and ready information. For example, inventory, complaints, and other information about storm water infrastructure (including pictures) can be placed in a database tied to geographic location.
- Complex problems or changes over time, such as water quality improvements, can be easily visualized in maps and graphs generated by GIS systems.GIS maps can be used to educate or convince citizens and political leadership concerning a course of action or a project's viability.
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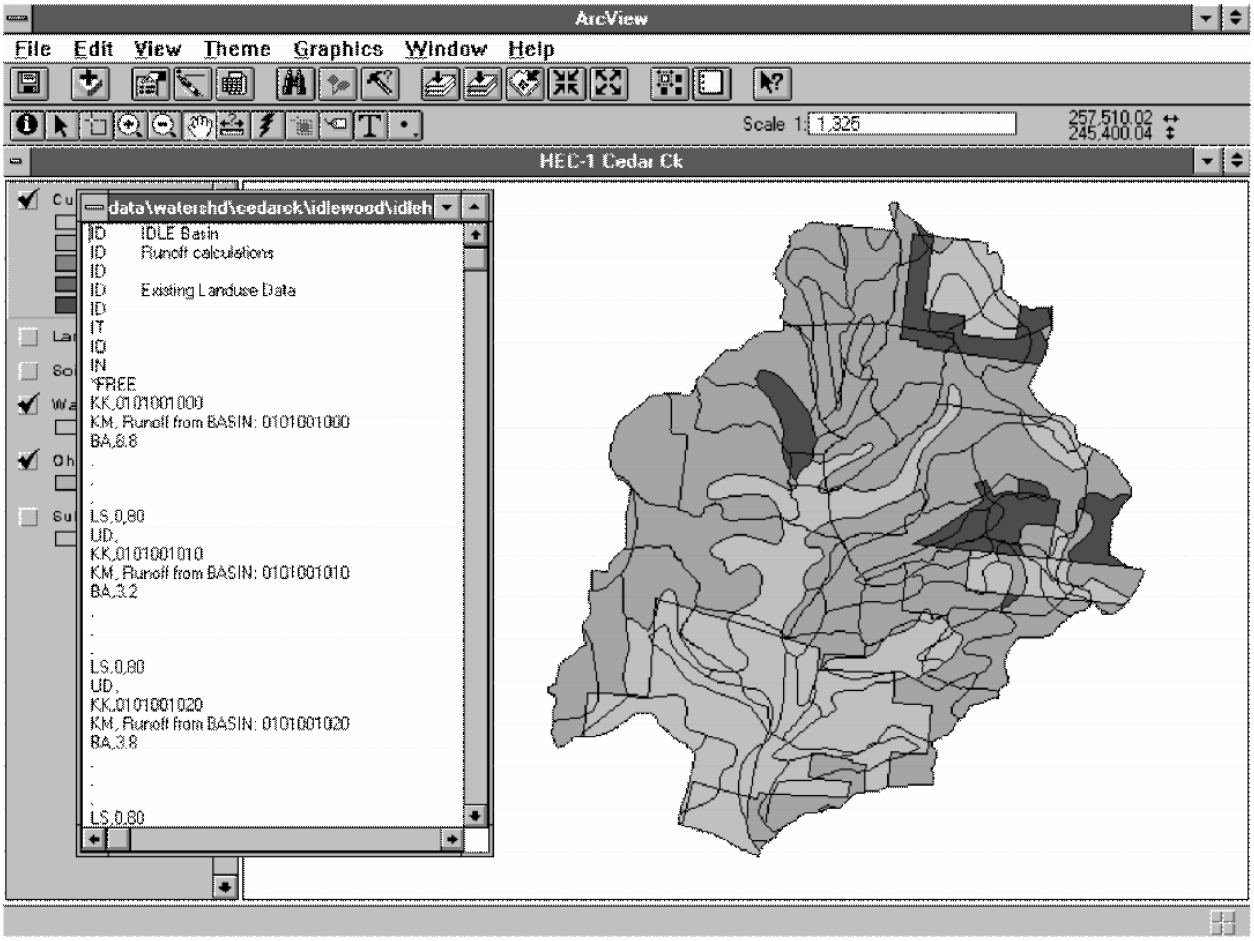

Figure G.5-2 Automated Hydrologic Modeling

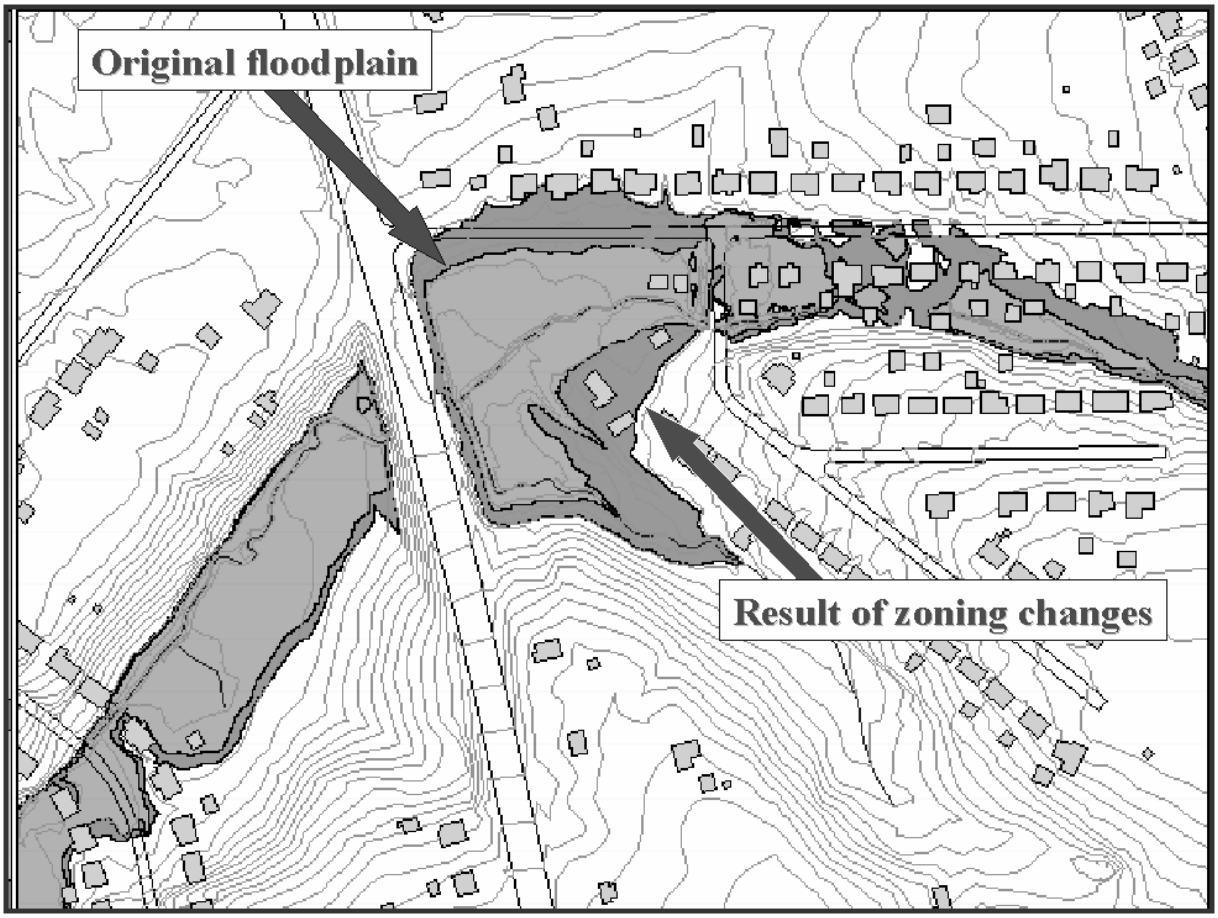

Figure G.5-3 Use of a GIS to Map Current and Potential Floodplains

Communities often make the mistake of making enormous expenditures on data, hardware, and software and databases but little on planning, staff familiarization, training, graphical user interface (GUI), and applications development. The end result is often an unusable system accessible by only a few who have the resources to learn the system, hire competent staff, and develop applications. It is often better to target the GIS implementation to certain needs and quickly roll-out applications that work for these needs even prior to the complete development of the database and overall system.

Proper implementation of GIS applications for storm water management involves planning for both "storm water only" applications and to integrate these applications with other potential uses within the municipality.

Other Related Technologies

GIS is closely related to several other types of information systems, and can be used with these other information tools, including:

- *Computer-Assisted Design (CAD)* – CAD systems evolved to create designs and plans of buildings and infrastructure. The systems are designed to do very detailed drafting and drawing but have only limited capability to attach data fields to the electronic drawing. As a result these systems do not have the capability to perform spatial analysis. Fortunately these drawings can be input to a GIS saving significant digitizing efforts. Once in a GIS, attribute data can then be added to the graphic features.

- *Database Management Systems (DBMS)* – These systems specialize in the storage and management of all types of data including geographic data. DBMSs are optimized to store and retrieve data and many GISs rely on them for this purpose. They do not typically have the analytic and visualization tools common to GIS.

- *Supervisory Control and Data Acquisition System (SCADA)* –These systems combine the ability to monitor information (e.g. rainfall, stream flow, flood level, etc.) remotely through telemetry. SCADA systems can also execute commands to do such things as open gates or close valves from a distance. Examples of the use of SCADA include automating storm water pump station operation, automated alarms for flood warning, and automated lowering of traffic control barrier arms during high water periods. SCADA systems can be combined with GIS to create comprehensive tracking systems.

G.5.2 Global Positioning Systems (GPS)

The Global Positioning System (GPS) is a space satellite based radio positioning system for obtaining accurate positional information for mapping or navigational purposes. GPS is made up of three distinct parts:

- *Satellites* – A constellation of satellites orbiting the earth continuously emit a timing signal, provided by on-board atomic clocks. The differential timing of satellite signals is used to calculate the distance from each satellite to the receiver.
- *Receiver* – A GPS receiver located on the ground converts satellite signals into position, velocity, and time estimates.
- *Ground Control* – The U.S. Department of Defense (DOD) developed and currently manages the maintenance of the satellite system. The DOD uses tracking antennas to constantly monitor the precise position of the NAVSTAR satellites. These positions can be used to correct for errors in the calculated positions of the roving receivers.

The GPS was built and is maintained by the U.S. government. The satellites orbit at an altitude of approximately 12,000 miles in a 12-hour pattern that provides coverage to the entire earth. The system is capable of serving an unlimited number of users free of charge.

A GPS receiver uses information from at least 4 satellites to precisely triangulate its position on the earth with about one meter accuracy. Special techniques can be used to obtain sub-meter accuracy. If a receiver cannot "see" four satellites, it can calculate a less-accurate estimate based on three satellites. Virtually all GPS receivers display basic positional information including latitude, longitude, elevation, and speed (if moving). Most receivers also display time, heading, the number of satellites in view, where those satellites are positioned in the sky, and signal quality. GPS receivers for data collection can collect both the location (coordinates) and the attribute data of a given geographical feature.

GPS Applications to Storm Water

Storm water infrastructure inventories can be conducted more easily and in far less time using GPS. In the past, traditional geodetic surveying was used to locate and map storm water system components. A transit survey requires traversing between a known point to the point of interest. GPS surveying is much more efficient, possibly taking as little as a few seconds to map each point. Using bicycle or car-mounted equipment, a community may be able to survey up to 500 points per day. Obviously, the users must be trained and knowledgeable of the GPS devices and the objectives of the inventory to insure accuracy of the data.

GPS inventory work can be integrated with GIS application software. For example, a GIS layer of structural control locations can be created using GPS data and linked to a maintenance database. GPS data can also be used in computer modeling activities for storm water management.

G.5.3 Remote Sensing

Remote Sensing is a technique for collecting observations of the earth using airborne platforms (airplanes and satellites) which have on-board instruments, or sensors. These sensors record physical images based on light, temperature, or other reflected electromagnetic energy. This sensor data may be recorded as either analog data, such as photos or digital image data. Figure G.5-4 gives an example of low (25-meter), medium (5-meter) and high (1-meter) resolution satellite imagery.

Figure G.5-4 Low, Medium and High Resolution Satellite Imagery
Source: Space Imaging, Inc.

Ground reference data is then applied to aid in the analysis and interpretation of the sensor data, to calibrate the sensor, and to verify the information extracted from the sensor data. Remotely sensed images have a number of advantages to on-the-ground observation, including:

- Remote sensing can provide a regional view.
- Remote sensing can provide repetitive looks at the same area over time.
- Remote sensors "see" over a broader portion of the electromagnetic spectrum than the human eye.
- Sensors can focus on specific bandwidths in an image and can also look at a number of bandwidths simultaneously.
- Remote sensors often record signals electronically and provide geographically referenced data in digital format.
- Remote sensors operate in all seasons, at night, and in bad weather.

The airborne platforms that carry remote sensing instruments can be any kind of aircraft or satellite observing the Earth at altitudes anywhere from a few thousand feet to orbits of hundreds of kilometers. Satellites may employ a variety of sensors for numerous of applications. Currently, no single sensor is sensitive to all wavelengths. All sensors have fixed limits of spectral sensitivity and spatial sensitivity, the limit on how small an object on the earth's surface can be seen. The common types of sensors aboard satellites include:

- *Multispectral Scanner (MSS) Sensors* – Data are sensed in as many as six bands including UV, visible, near-IR, mid-IR and thermal)
- *Thematic Mapper (TM) Sensors* – Data are sensed in seven spectral bands simultaneously including blue, green, red, near-infrared, and two in mid-infrared. The seventh band detects only the thermal portion of the spectrum.
- *Radio Detection and Ranging (RADAR)* – Examples are Doppler radar systems used in weather and cloud cover predictions.

The appropriate band or combination of MSS bands should be selected for each interpretive use. For example bands 4 (green) and 5 (red) are usually best for detecting cultural features such as urban areas, roads, new subdivisions, gravel pits, and quarries. The TM bands are more finely tuned for vegetation discrimination than those of the MSS due in part to the narrower width of the green and red bands.

Examples of the growing number of remote sensing satellites include the U.S. Landsat satellites, the Indian Remote Sensing (IRS) satellites, Canada's RADARSAT, and the European Space Agency's Radar Satellite. Images from these satellites have spatial resolutions ranging from approximately 100 meters to 15 meters or better. The first commercial satellite capable of resolving objects on the ground as small as one meter in diameter was recently launched. Several competing companies have similar offerings. For example, the IKONOS-2 features high spatial resolutions of 1-meter panchromatic (black and white) and 4-meter multispectral (color). Panchromatic data has a higher resolution while multispectral data provides better interpretation. Additionally, the 1-meter panchromatic spatial content can be combined with the spectral content of the 4-meter multispectral data. This 1-meter accuracy allows for a wide range of applications in storm water management at a price typically less than $500 per square mile (with some minimum order restrictions).

Digital Orthophoto Quarter Quadrangles (DOQQ's)

Orthophotos combine the image characteristics of a photograph with the geometric qualities of a map. Unlike standard aerial photography, relief displacement in orthophotos has been removed displaying ground features in their true ground position, thus allowing for direct measurement of distance, area, angles, and positions.

The National Aerial Photography Program (NAPP) is the primary source of aerial photography used in the production of 1-meter digital orthophotos. The North Central Texas Council of Governments (NCTCOG) maintains a database of digital aerial photography known as digital orthophoto quarter quadrangles (DOQQ's). More information can be found at www.nctcog.org/ris/gis/index.asp.

Figure G.5-5 illustrates a section of a DOQQ. Local government organizations that do not have access to their own digital aerial photography can acquire this data. The details concerning DOQQ's are:

- A standard DOQQ image covers an area of 3.75 x 3.75 minutes or ¼ of a USGS quadrangle with some overlap
- 1-meter pixel resolution (1-meter resolution provides the minimum resolution needed for capturing smaller impervious features within the 200 sq. ft. to 100 sq. ft. range)
- Projected in Universal Transverse Mercator (UTM) with units in meters
- Available in TIFF format
- Coverage for the entire state of Texas
- Images were captured between 1994 and 1997
- DOQQ images for all of North Central Texas can be obtained at the www.tnris.state.tx.us/digitaldata/doq.htm.

Figure G.5-5 Example DOQQ

Remote Sensing Applications for Storm Water

Satellite imagery offers a diverse set of mapping products for projects ranging from land use/land cover evaluation to urban and regional planning, tax assessment and collection, and growth monitoring. In the case of storm water runoff, multispectral imagery can be used to measure impervious surfaces, such as rooftops, streets, and parking lots. Pervious surfaces, such as tree- and grass-covered areas can also be measured or delineated. Applying runoff coefficients to the area of each surface type can provide the best available estimates for nonpoint source water pollution. By adding parcel boundaries, it is possible to provide estimates of runoff per parcel in order to calculate storm water user fees. Similarly, designated land use categories can be applied to the area of each surface type and in combination with the known soil coverage can be used to calculate hydrologic curve numbers. Flood boundaries can be measured within a few meters accuracy in areas without tree cover using submeter multispectral fused imagery. Individual buildings and parcel boundaries can also be identified in order to assess flood vulnerability.

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APPENDIX

H

Dams and Reservoirs in Texas

The Texas Commission on Environmental Quality (TCEQ) regulates the construction of dams in Texas that are six feet or more in height as per the Texas Administrative Code (Title 30, Part 1, Chapter 299 – Dams and Reservoirs). Approval from the TCEQ of plans and specifications is required for construction of a dam. The TCEQ also has the authority to inspect existing dams, and if necessary, require unsafe dams to be upgraded or removed. The Dam Safety Program is administered under the Field Operations Division of the Office of Compliance and Enforcement. Forms, Guidelines, Rules, Regulations, and many other resources can be found on-line at www.tnrcc.state.tx.us/enforcement/dam_safety/intro2.html.

H.1 Overview

Structures constructed for the purpose of the impounding water either on a temporary or permanent basis, which are over six feet in height are regulated by the State. The Texas Commission on Environmental Quality (TCEQ) is the regulatory agency responsible for administration of the State dam safety laws in Texas. Dams are classified according to size and the potential for loss of human life and/or property damage within the area downstream of the dam. The State regulates the design, construction, operation, and maintenance of dams, and this chapter provides an overview of some of the pertinent criteria.

H.2 Symbols and Definitions

The following words and terms, when used in this chapter, shall have the following meanings, unless the context clearly indicates otherwise:

Dam--Any barrier, including one for flood detention, designed to impound liquid volumes and which has a height of dam greater than six feet. This does not include highway, railroad, or other roadway embankments, including low water crossings that may temporarily detain floodwater, levees designed to prevent inundation by floodwater, closed dikes designed to temporarily impound liquids in the event of emergencies, or off-channel impoundments authorized by the commission in accordance with the Texas Water Code, Chapter 26, or the Texas Solid Waste Disposal Act, Texas Civil Statutes, Article 4477-7.

Effective crest of the dam--The elevation of the lowest point on the crest of the dam excluding spillways.

Existing dam--

(A) any dam constructed in accordance with necessary authorizations of the TCEQ;

(B) any existing dam exempt under the Texas Water Code, §11.142.

Height of dam--The vertical distance from the effective crest of the dam to the lowest elevation on the centerline or downstream toe of the dam, including the natural stream channel.

Maximum storage capacity--The volume of the impoundment created by the dam at the effective crest of the dam, usually expressed in acre-feet.

Normal storage capacity--The volume of the impoundment created by the dam at the lowest uncontrolled spillway crest, usually expressed in acre-feet.

Probable maximum flood (PMF)--The flood magnitude that may be expected from the most critical combination of meteorologic and hydrologic conditions that are reasonably possible for a given watershed.

Probable maximum precipitation (PMP)--Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year.

Proposed dam--Any dam, constructed or to be constructed, which is not included in the definition of existing dam.

Spillway design flood (SDF)--The flood criteria that needs to be considered in the design of a proposed project.

Spillway evaluation flood (SEF)--The flood criteria that needs to be considered in the hydrologic evaluation of an existing structure.

H.3 Dam Classifications

Texas Administrative Code, Chapter 299 – DAMS AND RESERVOIRS
In accordance with the Texas Administrative Code, Chapter 299, the State of Texas considers a dam any barrier designed to impound liquid volumes that has a height greater that six feet. Dams in Texas are classified based on their size (height), hazard potential (failure consequences), and hydrologic criteria (spillway capacity).

H.3.1 Size

The Texas Administrative Code classifies the size of a dam as small, intermediate, or large depending upon the height of the dam and the storage volume of the reservoir or pond created by the dam. The height of a dam is the vertical distance from the lowest point on the dam crest, excluding the spillway, to the lowest elevation on the centerline or downstream toe of the dam, including the natural stream channel. The storage capacity in acre-feet is the volume of the pond when the water level is at the crest of the service spillway. A “rule of thumb” is that the storage capacity of a pond in acre-feet is equal to the surface area of the pond in acres times 40 percent of the maximum depth of the pond in feet. Once the dam height and pond volume are known, the size classification of the dam is determined by the following table:

SIZE CLASSIFICATION		
Category	Storage (Ac-Ft) Impoundment	Height (Ft.)
Small	Less than 1,000	Less than 40
Intermediate	Equal to or Greater than 1,000 & less than 50,000	Equal to or Greater than 40 & less than 100
Large	Equal to or Greater than 50,000	Equal to or Greater than 100

H.3.2 Hazard Potential

Hazard classification pertains to potential loss of human life and/or property damage within either existing or potential developments in the area downstream of the dam in event of failure or malfunction of the dam or appurtenant facilities. Hazard classification does not indicate any condition of the dam itself. Dams in the low hazard potential category are normally those in rural areas where failure may damage farm buildings, limited agricultural improvements, and county roads. Significant hazard potential category dams are usually those in predominantly rural areas where failure would not be expected to cause loss of human life, but may cause damage to isolated homes, secondary highways, minor railroads, or cause interruption of service or use (including the design purpose of the facility) of relatively important public utilities. Dams in the high hazard potential category are usually those in or near urban areas where failure would be expected to cause loss of human life, extensive damage to agricultural, industrial, or commercial facilities, important public utilities (including the design purpose of the facility), main highways, or railroads.

The Texas Administrative Code's hazard classifications for dams include the classifications of *low, significant or high* according to the criteria listed in the following table:

HAZARD POTENTIAL CLASSIFICATION		
Category	Loss of Human Life	Economic Loss
Low	None expected (No Permanent structures for human habitation)	Minimal (Underdeveloped to occasional structures or agricultural improvements)
Significant	Possible, but not expected (A small number of inhabitable structures)	Appreciable (Notable agricultural, industrial or commercial development)
High	Expected (Urban development or large number of inhabitable structures)	Excessive (Extensive public, industrial, commercial or agricultural development)

H.3.3 Hydrologic Criteria

Uncontrolled floodwaters are one of the most powerful and destructive forces in nature. Dams that are not designed to withstand major storms may be destroyed by them, increasing flood damage downstream. This damage is too often catastrophic. In order to protect lives and property downstream, the Texas Administrative Code requires that dams be constructed to safely handle an appropriate percentage of the Probable Maximum Flood (PMF). This percentage varies according to the height of the dam, size of the impoundment, and extent and severity of damage possible upon failure. The requirements established in Texas are similar to those used in other states, and historical records of significant storms and dam failures this century verify that the design criteria are reasonable.

H.3.4 Development of the PMP

Scientists use both meteorological methods and historical records to determine the greatest amount of precipitation that is theoretically possible within a region. The historical data consists of point precipitation amounts measured at rain gages throughout the region being studied or a region with very similar meteorologic and topographic characteristics. These rainfall data are subsequently maximized through "moisture maximization" and other numerical methods. Moisture maximization is a process in which the maximum possible atmospheric moisture for a region is applied to rainfall data from a historic storm. This process increases the rainfall depths, bringing them closer to their potential maximum.

Probable maximum precipitation amounts vary slightly throughout Texas because of variations in topography and meteorology. The PMP is greatest in the eastern portion of the state. Furthermore, not

all storms have the same duration. Using the methods mentioned above, the PMP has been determined for different storm periods, generally ranging from six to seventy two hours.

H.3.5 Development of the PMF

The Probable Maximum Flood is the flood that is a direct result of the Probable Maximum Precipitation. However, drainage areas with the same PMP may have different PMFs. This is possible because the amount of flooding which results from a given rainfall amount depends upon the characteristics of the drainage basin. For this reason, the PMF, not the PMP, must be used as a design criterion for a dam. Some important characteristics include soil type, land use, size and shape of the watershed, and average watershed slope. Both the volume and rate of runoff are affected. For example, water will run off of steep slopes more quickly than gentle ones. More water will infiltrate sandy soils than clay.

H.3.6 Application of the PMF

The percentage of the PMF that a dam must be designed to withstand depends upon its classification. The Texas Administrative Code's hydrologic criteria for dams are shown in the table below.

HYDROLOGIC CRITERIA FOR DAMS		
Classification		-
Hazard	Size	Minimum Flood Hydrograph
Low (No. 3)	Small	1/4 PMF
	Intermediate	1/4 PMF to 1/2 PMF
	Large	PMF
Significant (No. 2)	Small	1/4 PMF to 1/2 PMF
	Intermediate	1/2 PMF to PMF
	Large	PMF
High (No. 1)	Small	PMF
	Intermediate	PMF
	Large	PMF

The exact percent of the PMF to be passed is determined by a straight-line interpolation of the dam height in the range. For example, a small, significant hazard dam would be between 6 to 40 feet high. If the dam has a height of 20 feet, the hydrologic criteria would be that the dam spillway should pass 25% + (20'-6') x (50%-25%) / (40'-6') or 35.3% of the PMF.

There are some exemptions to minimum hydrologic criteria. Proposed low hazard dams exempt under the Texas Water Code, §11.142, (see below) are exempt from the minimum criteria. Any other proposed structure may be exempt from the minimum criteria if properly prepared dam breach analyses show that existing downstream improvements or known or planned future improvements will not be adversely affected. A properly prepared breach analysis should include at least three events, the normal storage capacity nonflood event, the barely overtopping event, and the PMF event. Data on additional flood magnitudes may be provided as necessary to document other conditions or conclusions. Downstream flooding differentials of one-foot or less between breach and nonbreach simulations are not considered to be adverse.

H.3.7 Water Rights

The Texas Water Code requires anyone appropriating State Water or beginning work to store or divert State Water to obtain a permit from the Texas Commission on Environmental Quality (TCEQ). Private developments with ponds that contain 200 acre-feet or less of storage and that are used exclusively for domestic and livestock purposes are exempt from this requirement (11.142–Permit Exemptions). This exemption is based in part on the premise that all such ponds would by their nature of construction and due to their location on ranches or farms, be *low* hazard dams. However, as soon as the purpose for and use of these ponds change, there is a requirement to inform the TCEQ and obtain a water rights permit. Usually the downstream water rights holders do not challenge these new storage permits for existing small ponds; however notifications are sent out and public hearings could be held if there are objections to the permit. All dams that are classified as *significant* or *high* hazard must have a permit.

If any other use of the stored water were contemplated, such as using the pond water for irrigation purposes, then another water right authorization would be required. Authorization for a permit to divert State Water for other than domestic and livestock uses from a reservoir permitted for storage only should be requested pursuant to Texas Water Code, 11.143.

H.4 Design Criteria

Preparation of all plans and specifications for the construction, enlargement, alteration, repair, or removal of dams subject to TCEQ review shall be under the supervision of an engineer licensed in Texas. Construction of a dam or the enlargement, repair, or alteration of an existing dam requiring TCEQ authorization shall not be commenced prior to the executive director's written approval of final construction plans and specifications. Construction plans and specifications shall be submitted to the executive director and shall be as completely detailed as necessary for submission to the contractors bidding on the proposal. The proper design of a dam involves a complex combination of engineering applications.

Failure of an impoundment structure and the sudden release of water is referred to as a dam breach. A breach may occur during flooding or in fair weather due to effects of erosion or seepage. The hydrologic and hydraulic design determines what the potential impacts of a dam breach might be and what volume and flow rate a structure must withstand.

The geotechnical and structural design must consider that the structure can safely withstand the hydraulic forces imposed by the estimated hydrologic and hydraulic design. The following steps are necessary for the design of an embankment or concrete dam and each step must be thoroughly evaluated and documented.

1. Hydrologic and Hydraulic Design

   Determination of controlling design conditions and associated storm runoff.

   Setting of spillway and stilling basin widths and elevations, top of dam elevation, and normal pool elevation if necessary.
2. Structural and geotechnical design of embankments, concrete structure, spillways, and drawdown structures.
3. Development of operation and maintenance procedures.

H.4.1 Hydrologic and Hydraulic Design

The height of the dam, the amount of freeboard, and the size and capacity of the spillways must be designed to balance the hydrologic and hydraulic properties. The structure must be designed based on the size and classification of the dam. The hydrologic and hydraulic modeling is discussed elsewhere in the manual. General considerations include the sizing of the capacity of the outlet works and spillway.

H.4.2 Geotechnical Considerations

Information concerning geology, seismicity, foundation conditions, and construction materials is essential for any size structure. It is necessary for the stability of the structure that the material used in the embankment, the foundation, and adjacent earth have the necessary structural properties to withstand the hydrostatic forces required by the design. The foundation for these structures may vary significantly, and will affect the general shape and configuration of the embankment. In addition, the potential for seepage should be identified and provided for in the design. The embankment should also be adequately protected from surface erosion.

Field investigations to determine the soils and groundwater conditions for stability and seepage control must be considered. Stability analysis should consider after construction conditions based on shear strength parameters as determined by laboratory tests.

H.4.3 Structural Considerations

Structural designs should be prepared in accordance with generally accepted structural engineering practices. Components of the dam, spillway, or outlet structures should resist the most critical loading considering buoyancy, sliding, hydrostatic uplift, overturning, soil and water pressure forces, impact forces, and other loads.

Embankment dams are the most common type of dam constructed today. They generally have a trapezoidal shape with side slopes of two horizontal to one vertical or flatter. The ability of an embankment dam to resist the hydrostatic pressure caused by the water in the reservoir or detention pond is primarily the result of the weight of the materials and the strength of the materials which are used.

The embankment must be stable and resistant to movement under all operating conditions including rapid drawdown. Seepage through the embankment, abutments, and foundation should be controlled so that piping or surface slides do not occur. The embankment must be safe form overtopping by flow and surface erosion, or protected to allow these flows. The embankment must be safe from damage due to wave action or rain.

Concrete structures use many of the same considerations in the design. Concrete dams generally use their own weight as the primary resistance to sliding. Other considerations include the strength of the concrete, the hydrostatic uplift on the structure, silt load which may accumulate on the upstream toe, and overturning and impact forces.

H.4.4 Seepage Considerations

Embankment materials vary with the site. Their water retention capability is due to the low permeability of the embankment material whether it is a homogeneous mass of material or a zone of low permeability material in the structure. Materials used for the construction of embankment dams can include soil or rock. If the difference between the permeability of the core section of the embankment and the downstream shell is great, an internal drainage system is probably not needed. If the variation in permeability is not great, then the entire embankment section can become saturated and that condition reduces the stability of the structure. The drainage system could consist of a chimney drain, a toe drain, a blanket drain, finger drains, or a combination of these items.

Concrete structures are also subject to seepage. Seepage can be present at the dam foundation interface reducing the sliding resistance. A line of drain holes away from the upstream face can relieve seepage pressure under the structure.

H.4.5 Other Considerations

Slopes for most embankment sections can be as steep as two horizontal to one vertical, however if the slope is designed with grass and will need to be mowed, a three horizontal to one vertical is about as steep as can be maintained without special equipment. The higher the plasticity index of the material in the embankment section, the greater the possibility for slope failures; therefore, flatter slopes should be used. Compaction of materials in the construction process is another critical item for consideration. The need for rock riprap or other type of slope protection is usually evaluated on a case specific need depending on the potential for wave or water erosion. Grass on the slopes, particularly the downstream slope, generally provides adequate protection for structures where overtopping is not an issue.

Slopes for concrete structures, especially if they are designed for overtopping, need to consider the difference between headwater and tailwater, the potential for a hydraulic jump, and overturning stability.

There are many resources available for the design of dams. Design of Small Dams by the Bureau of Reclamation is a good resource for both embankment structures and concrete dams.

The forms and checklist included after Section H.5 must be submitted to the TCEQ to obtain approval for a dam.

H.5 Operation and Maintenance

Once completed, the structure must be kept in good condition by regular maintenance. This includes periodic inspection of the structure. Evaluating the structural condition of an existing dam includes, but is not limited to, visual inspections and evaluations of potential problems such as seepage, cracks, slides, conduit and control malfunctions, erosion, rodent damage, spillway blockage, and other structural and maintenance deficiencies which could lead to failure of a structure. In addition, existing dams may need to be re-evaluated in consideration of continuing downstream development.

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TCEQ

TEXAS DAM SAFETY PROGRAM
DIVISION OF FIELD OPERATIONS - OFFICE OF COMPLIANCE & ENFORCEMENT
12100 PARK 35 CIRCLE BUILDING A, MC-174
Austin, Texas 78753

ENGINEER'S NOTIFICATION OF COMPLETION OF CONSTRUCTION, MODIFICATION, REPAIR OR REMOVAL OF A DAM

(PLEASE PRINT OR TYPE)

PLEASE CHECK ONE: New___, Modification___, Repair___, Removal___, Rebuild

(Please see 30 TAC 299 Rules for Dam Safety and Dam Construction, and attach the AAs-Built@ drawings)

NAT ID: _____ County: _____

Adjudication Number: _____ Region: _____

Name of Dam: _____

Name of Project:_____

Owner:

Name _____, Street or P.O. Box _____

City _____, State TX_____, Zip Code _____

Phone Number ( )_____, Emergency Contact Phone ( ) _____

Fax Number ( )_____ Email _____

Engineering Firm:

Engineering Firm Name _____

Engineer Name _____, Title_____,TX P.E. License No _____

Engineering Firm Address _____

City _____, State _____ Zip Code _____

Phone ( )_____, Fax ( ) _____, Email _____

The project was completed on _____, 20____. To the best of my knowledge, the Project was constructed in substantial conformance with plans, specifications, and change orders filed with the approved by the Texas Commission on Environmental Quality.

_____

name. P.E.

Form DS-1005-Cert

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TEXAS DAM SAFETY PROGRAM
DIVISION OF FIELD OPERATIONS - OFFICE OF COMPLIANCE & ENFORCEMENT
12100 PARK 35 CIRCLE BUILDING A, MC-174
Austin, Texas 78753

APPLICATION FOR REVIEW OF PLANS AND SPECIFICATIONS FOR THE NEW CONSTRUCTION, MODIFICATION, REPAIR OR REMOVAL OF A DAM

(PLEASE PRINT OR TYPE APPLICATION)

PLEASE CHECK ONE: New___, Modification___, Repair___, Remove__, Rebuild____

(SEE 30 TAC 299 Rules for Dam Safety and Dam Construction)

SECTION 1: OWNER INFORMATION

I4, _____, owner, hereby accept and approve the enclosed plans
(Name of Legal Owner)

and specifications for submittal to the TCEQ Dam Safety Program in accordance with 30 TAC Chapter 299. I certify that I will notify TCEQ immediately any changing from this application.

_____
(Signature of Owner) (Date)

Street or P.O. Box: _____

City _____, State _____, Zip Code _____

Phone Number ( ) _____, Emergency Contact Phone ( ) _____,

Fax Number ( ) _____ Email _____

Owner Code (Please check one): Federal (F), ____ Local Government (L), _____ Utility (U), _____ Private (P) _____,

State(S), _____, Others (O), please specify _____

Dam and Reservoir Use: _____, Year to build _____

(Augmentation, Diversion, Domestic, Erosion Control, Evaporation, Flood Control, Fire Control, Fish, Hydroelectric, Industrial, Irrigation, Mining, Municipal, Pollution Control, Recreation, Stock water, Settling Ponds, Tailings, Waste Disposal)

Engineering Firm _____

Engineer Name _____, Title _____, Texas P.E. License Number_____

Engineering Firm Address _____

City _____, State _____, Zip _____

Phone ( )_____, Fax ( )_____, Email _____

Page 1 of 3

SECTION 2: GENERAL INFORMATION

Name of Dam _____

Other Name(s) of Dam _____

Reservoir Name _____

Location: Texas _____ Region _____, Latitude _____, Longitude _____,

County _____, Stream Name: _____, Tributary _____,

River Basin _____, Topographic Map no. _____,

Nearest City/Town _____, Distance from Nearest City/Town _____,

TX Number (if known) _____, Water Rights Number (if known) _____,

Date of Emergency Action Plan (EAP), if having _____

SECTION 3: RESERVOIR & DAM STRUCTURES

Reservoir:

Large _____, Medium _____, Small _____, High Hazard _____, Significant Hazard _____, Low Hazard _____,

No. of Population at Risk _____, Specify if needed _____

Type of Dam – (e.g. Earthfill, Homogeneous):

Concrete _____ Gravity _____ Arch _____

Earthfill _____ Zoned _____ Homogeneous _____

Rockfill _____ Masonary _____ Impervious Membrane _____

Other (Specify) _____

Dam Structure: (Dimensions to nearest tenth of foot, volume to nearest acre-feet or cubic-yard, areas to nearest acre):

Jurisdictional Height _____ ft. (Natural surface of ground to bottom of emergency spillway at longitudinal centerline)

Embankment Height _____ ft. (Jurisdictional height plus emergency spillway freeboard)

Structural Height _____ ft. (Bottom of cutoff trench to crest of dam at centerline)

Crest Length _____ ft. Crest Width _____ ft, Crest Elev. _____ ft. M.S.L.,

Pool Elevation _____ ft. M.S. L., Principal Elevation _____ft. M.S.L., Emergency Elevation _____ft. M.S.L.,

Top Dam Elevation _____ ft. M.S.L., Embankment Volume _____C.Y.

Maximum Impoundment Capacity _____ Acre-Feet (to crest of dam)

Normal Reservoir Capacity _____ Acre-Feet (at high waterline)

Reservoir Surface Area _____ Acres (at high waterline)

Page 2 of 3

Upstream (U)/Downstream (D), Facing Material (Place U or D as appropriate):

Concrete Dam _____, Concrete Facing _____, Clay _____, Gabions _____, Gravel _____,

Hand placed riprap _____, Masonry Dam _____, Natural _____, Planted _____,

riprap with bedding _____, riprap with no bedding _____, Rock Zone _____, Soil Cement _____, Steel _____,

Wood _____, Other? (Describe) _____

Outlet:

Outlet Diameter:_____Inches/Feet, Type:

Maximum Discharge Capacity: _____ cfs (Reservoir at high waterline).

Principal Spillway:

Material: _____ (i.e., Natural, Riprap, Concrete, CMP, RCP, etc.,)

Width (Diam): _____ ft.,Freeboard: _____ ft., Capacity: _____ cfs

Emergency Spillway:

Material: _____ (i.e., Natural, Riprap, Concrete, CMP, RCP, etc.,)

Width (Diam): _____ ft.,Freeboard: _____ ft., Capacity: _____ cfs

Total Spillway Capacity: _____ cfs, (Crest of the dam)

SECTION 4: HYDROLOGIC DATA *

Full PMF _____, 0.75PMF _____, 0.50PMF _____, 0.25PMF _____, Other_____

Drainage Area: _____Acres, or _____ Sq. Miles

Inflow Design Flood: _____, (i.e., 100-year %PMP, etc.) Duration _____ Hrs.

Type: _____, (i.e., Thunderstorm, General, Snowmelt, Combination)

Data Source Ref.

Peak Discharge _____ cfs; Volume _____ A., F., Basin lag time (Tc): _____ Hrs.

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Form DS-1001-A

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HYDROLOGIC AND HYDRAULIC (H&H) EVALUATION SUMMARY

(Please complete all sections , or otherwise specified)

Name of Dam:
TCEQ Dam Safety Project No.:
County:
Year to Build:

Maximum Record Precipitation (Inches):
Record Area (County/City):
Duration (Hrs):
Date of Record (MM/DD/YY):
Source Ref. (FEMA, National Weather...):

Downstream Dam Toe (ft-msl)		Normal Reservoir Capacity (ac-ft)	
Normal Pool (ft-msl)		Maximum Reservoir Capacity (ac-ft)	
Principal Spillway (ft-msl)		Reservoir Surface Area (ac)	
Emergency Spillway (ft-msl)		Drainage Area (ac)	
Top of Dam (ft-msl)		Outlet Diameter (In)	

Storm	Peak Inflow (cfs)	Peak Outflow (cfs)	Max. Stage (ft-msl)	Comments (if needed)
100-Yr				
25%PMF				
50%PMF				
75%PMF				
100%PMF				
PMF Breach				

To the best of my knowledge, I certify the above data is correct. I will provide the Hydrologic and Hydraulic Reports to Texas Commission on Environmental Quality upon request.

(Seal)

_____
(Signature & Date)

Form DS-1007-HH

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