Planning:

1.0 *integrated* Planning and Design Focus Areas
2.0 *integrated* Site Design Practices

Table of Contents

List of Tables

List of Figures

1.0 *integrated* Planning and Design Focus Areas

1.1 Introduction

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

- Remove pollutants in stormwater runoff to protect water quality;
- Regulate discharge from the site to minimize downstream bank and channel erosion; and
- Control runoff within and from the site to minimize flood risk to people and properties for the conveyance storm, as well as the 100-year storm.

The *integrated* Design Focus Areas are a coordinated set of design standards that allow the site engineer to design and size stormwater controls to address these goals. Each of the *integrated* Design Focus Areas should be used in conjunction with the others to address the overall stormwater impacts from a development site. When used as a set, the *integrated* Design Focus Areas control the entire range of hydrologic events, from the smallest runoff-producing rainfalls up to the 100-year, 24-hour storm. Through the *integrated* Design Focus Areas, each community receives standardized options while retaining the flexibility to define their own program. The *iSWM Criteria Manual for Site Development and Construction (Criteria Manual)* specifies the options allowed and/or required by the community.

The design focus areas for each of the goals above is summarized in Table 1.1 below:

Table 1.1 Summary of Options for Design Focus Areas

Design Focus Area	Criteria Manual Reference Section	Required Downstream Assessment	Design Options
Water Quality Protection	3.2	no	**Option 1:** Use *integrated* Site Design Practices for conserving natural features, reducing impervious cover, and using the natural drainage systems
			Option 2: 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)
			Option 3: Assist in implementing off-site community stormwater pollution prevention programs/activities as designated in an approved stormwater master plan or TPDES Stormwater permit
Streambank Protection	3.4	yes	**Option 1:** Reinforce/stabilize downstream conditions
			Option 2: Install stormwater controls to maintain or improve existing downstream conditions
			Option 3: Provide on-site controlled release of the 1-year, 24-hour storm event over a period of 24 hours (Streambank Protection Volume, SP_V)
Flood Mitigation and Conveyance	3.5 and 3.6	yes	**Flood Mitigation** **Option 1:** Provide adequate downstream conveyance systems
			Option 2: Install stormwater controls on-site to maintain or improve existing downstream conditions
			Option 3: In lieu of a downstream assessment, maintain existing on-site runoff conditions
			Conveyance Minimize localized site flooding of streets, sidewalks, and properties by a combination of on-site stormwater controls and conveyance systems

1.2 Downstream Assessment

As part of the iSWM 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 Focus Areas 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 that 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
    - Streambank protection storm
    - Conveyance storm
    - Flood mitigation storm
- Separate analysis for each major outfall from the proposed development

Once the analysis is complete, the designer should ask the following four 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?
- Are the post-development flood heights more than 0.1 feet above the pre-development flood heights?

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 1.0 and 2.0 of the Hydrology Technical Manual* gives additional guidance on calculating the discharges and velocities, as well as determining the downstream extent of the assessment.

1.3 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 stormwater runoff from development. A community should provide adequate water quality protection for development sites by specifying in the *Criteria Manual* the acceptable *integrated* Site Design Practices for the local community. Water quality protection shall only be required as identified by the *Criteria Manual*. Enhanced water quality protection can be achieved by using one or both of Options 2 and 3.

<u>Option 1: Use of *integrated* Site Design Practices</u>

Through the consideration and use of *integrated* Site Design Practices, as discussed in *Section 2.0* below, natural drainage and treatment systems can be preserved. With conservation of natural features, reduced imperviousness, and the use of natural drainage systems, the generation of stormwater runoff and pollutants from the site are reduced.

Option 2: 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 *Criteria Manual*, stormwater 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 1.3 of the Water Quality Technical Manual*.

Option 3: 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 stormwater 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.

1.4 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 three options by which a community can provide adequate streambank protection downstream of a proposed development. The local jurisdiction should specify in the *Criteria Manual* 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 *Criteria Manual*.

Option 1: Reinforce/Stabilize Downstream Conditions

If the increased velocities are higher than the allowable velocity of the downstream receiving system, then the developer must reinforce/stabilize the downstream conveyance system. The proposed modifications must be designed so that the downstream post-development velocities (for all storm events required by the municipality) 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 3.2 and 3.3 in the Hydraulics Technical Manual* for allowable velocities.)

Option 2: Install Stormwater 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 storm events required by the municipality are within an allowable range once the controls are installed.

Option 3: Control the Release of the 1-yr, 24-hr Storm Event

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 (SPv)

SP_v Calculation Methods: Several methods can be used to calculate the SP_v storage volume required for a site. *Section 3.0 of the Hydrology Technical Manual* illustrates the recommended average outflow method for volume calculation.

Hydrograph Generation: The SCS TR-55 hydrograph methods provided in *Section 1.3 of the Hydrology Technical Manual* 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 *Section 5.0 of the Hydrology Technical Manual* 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 stormwater 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 stormwater 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 2.2 of the Hydraulics Technical Manual*). Clogging protection must be provided on all orifices.

1.5 Flood Control

Flood control analyses are based on the following three (3) storm events. The storm frequencies for each event shall be established in *Section 1.3 of the Criteria Manual*.

- Streambank Protection
- Conveyance
- Flood Mitigation

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 "Flood Mitigation" storm events; maintain the boundaries of the mapped 100-year floodplain; and protect the physical integrity of the on-site stormwater controls and the downstream stormwater and flood control facilities.

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

1.5.1 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 stormwater 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 *Criteria Manual*.

It is recommended that once the initial set of controls are selected in the iSWM Site Plan design, the full build-out Flood Mitigation (100-year, 24-hour) storm be routed through the on-site conveyance system and stormwater 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 Flood Mitigation 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 Flood Mitigation storm events, traffic may be limited in order to utilize all or portions of the right-of-way for stormwater conveyance in order to protect properties. As such, the effective management of stormwater throughout the development for the full range of storm events is needed.

1.5.2 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 "Flood Mitigation" storm events (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 *Criteria 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 *Criteria Manual*.

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 three options by which a community can address downstream flood control. The local jurisdiction should specify in the *Criteria Manual* which of these options are acceptable, as well as any other alternatives for downstream flood control. These options closely follow the three options for Streambank Protection.

<u>Option 1: Provide Adequate Downstream Conveyance Systems</u>

If the downstream receiving system does not have adequate capacity, then the developer shall 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 stormwater 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.

<u>Option 2: Install Stormwater Controls to Maintain Existing Downstream Conditions</u>

If the downstream receiving system does not have adequate capacity, then the developer shall provide stormwater 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.

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.

<u>Option 3: In lieu of a Downstream Assessment, Maintain Existing On-Site Runoff Conditions</u>

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 3 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 2.0 of the Hydrology Technical Manual*. The developer shall confirm that detention does not exacerbate peak flows or create a worse situation in downstream reaches.

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

- *Peak-Discharge and Hydrograph Generation:* Hydrograph methods provided in *Section 1.0 of the Hydrology Technical Manual* 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 *Section 5.0 of the Hydrology Technical Manual* 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 2.0 of the Hydrology Technical Manual.*

1.6 *integrated* Watershed Planning

1.6.1 Introduction

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

Stormwater 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 stormwater 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 stormwater master plan is the comprehensive watershed plan which is described in detail in this chapter.

For any stormwater 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.

1.6.2 Types of Stormwater Master Planning

There are several basic types of stormwater master plans that can be prepared. The *Criteria Manual* 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 stormwater 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 stormwater 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 stormwater 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. Stormwater 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 stormwater 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 stormwater 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 stormwater 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.

2.0 *integrated* Site Design Practices

2.1 Overview

2.1.1 Introduction

The first step in addressing stormwater 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 stormwater 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 stormwater (quantity and quality) as close to the point of origin as possible and minimizing collection and conveyance
- Preventing stormwater impacts rather than mitigating them
- Utilizing simple, nonstructural methods for stormwater 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 stormwater controls

Integrated site design for stormwater 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 stormwater 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 stormwater 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 stormwater 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 stormwater 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 3.2.2 of the Criteria Manual* describes the 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.

The use of stormwater *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

2.1.2 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 Stormwater "Islands"
- **Utilization of Natural Features for Stormwater 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 *Section 2.2*. 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.

The *integrated* Site Design Practices may be subject to other ordinances within a municipality and could require approval before implementation. Review all relevant materials before developing a site plan.

2.1.3 Using *integrated* Site Design Practices

Site design should be done in unison with the design and layout of stormwater infrastructure in attaining stormwater management goals. Figure 2.1 illustrates the *integrated* site design process that utilizes the four *integrated* site design categories.

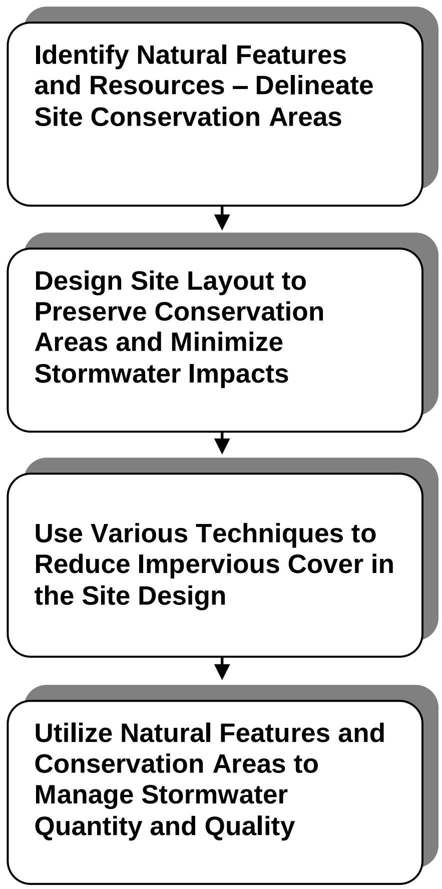


Figure 2.1 *integrated* Site Design Process

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 stormwater quantity and quality management purposes.

2.2 *integrated* Site Design Practices

2.2.1 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 stormwater runoff, providing runoff storage, reducing flooding, preventing soil erosion, promoting infiltration, and removing stormwater 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 *Section 2.2 of the Criteria Manual*). From this site analysis, a concept plan for a site can be prepared that provides for the conservation and protection of natural features. Figure 2.2 shows an example of the delineation of natural features on a base map of a development parcel.

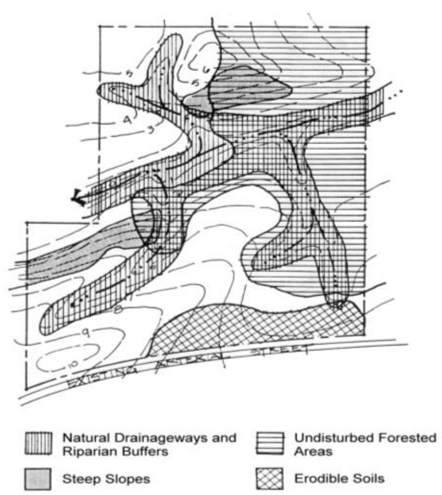


Figure 2.2 Example of Natural Feature Delineation
(Source: MPCA, 1989)

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 stormwater filtering and infiltration zones • Helps to preserve the site's natural character and aesthetic features • May increase the value of the developed property • A stormwater 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 stormwater 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 2.3 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.

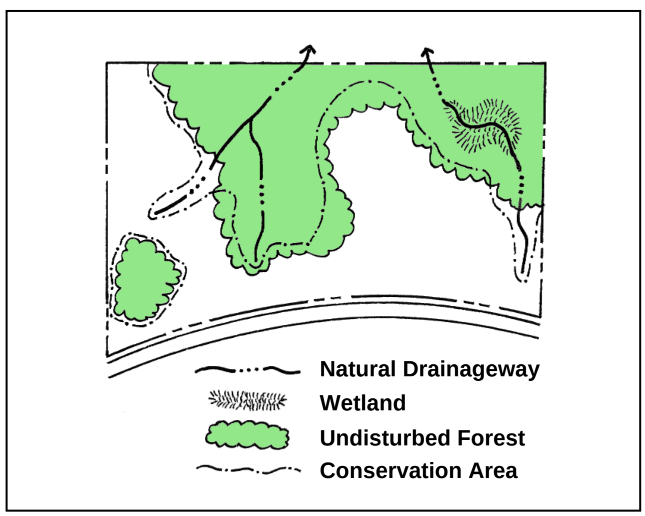


Figure 2.3 Delineation of Natural Conservation Areas

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 stormwater 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 stormwater 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 stormwater 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 2.4

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 stormwater to concentrate and flow into the stream without first flowing through the buffer.

Figure 2.4 Riparian Stream Buffer

Ideally, riparian buffers should be sized to include the flood mitigation storm 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 2.5. The function, vegetative target and allowable uses vary by zone as described in Table 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 supersede 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.

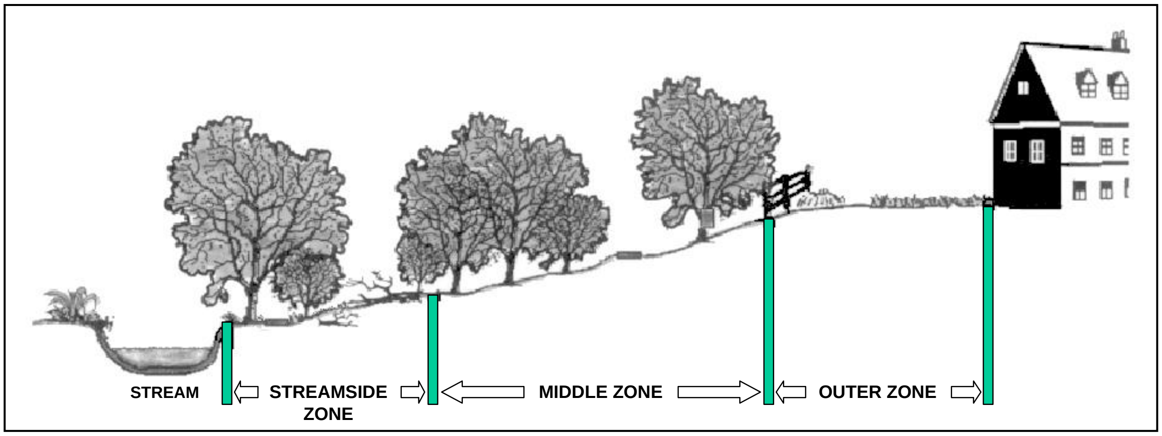


Figure 2.5 Three-Zone Stream Buffer System

Table 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 flood mitigation storm 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 stormwater controls, bike paths	**Unrestricted** e.g., residential uses including lawn, garden, most stormwater 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 flood mitigation storm 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 stormwater, 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.

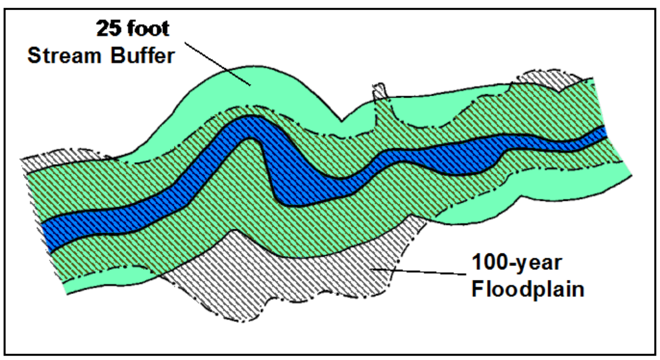


Figure 2.6 Floodplain Boundaries in Relation to a Riparian Buffer

As such, floodplain areas should be avoided on a development site. Ideally, the entire flood mitigation storm 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, flood mitigation storm floodplain boundaries may lie inside or outside of a preserved riparian buffer corridor, as shown in Figure 2.6.

Maps of the flood mitigation storm 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 stormwater 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 stormwater 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 2.7.

Therefore, development on slopes with a grade of 15% or greater should be avoided if possible to limit soil loss, erosion, excessive stormwater 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.

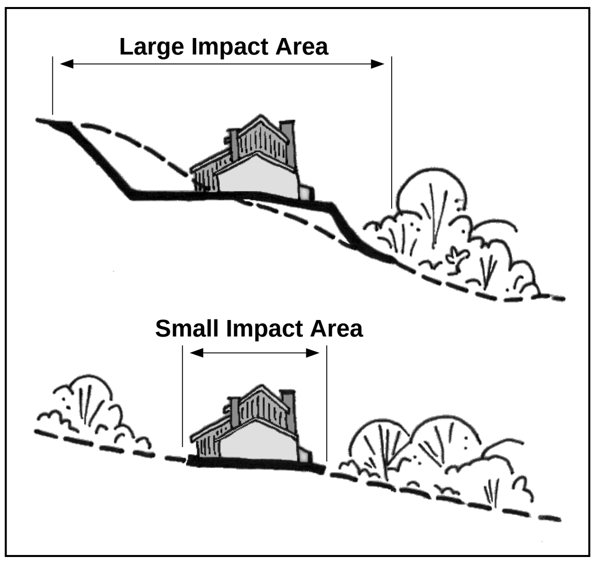


Figure 2.7 Flattening Steep Slopes for Building Sites Uses More Land Area than Building on Flatter Slopes
(Source: MPCA, 1989)

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 stormwater runoff and should be preserved as a potential stormwater 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 stormwater infiltration zones. A stormwater 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 stormwater 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 2.8. Soil surveys can provide a considerable amount of information relating to all relevant aspects of soils. *Section 6.0 of the Hydrology Technical 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.

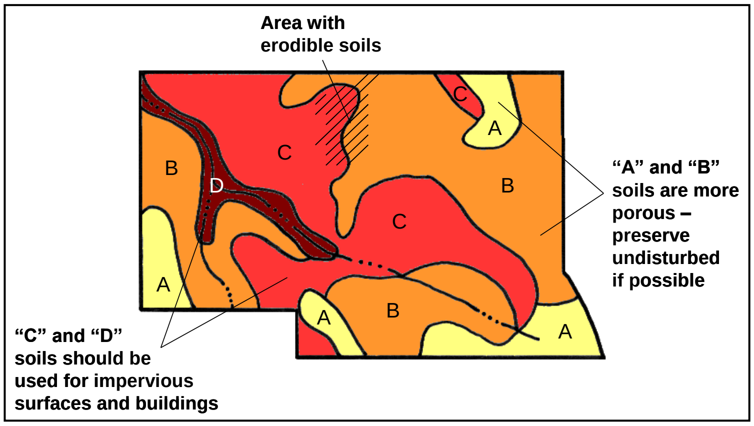


Figure 2.8 Soil Mapping Information Can Be Used to Guide Development

2.2.2 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 stormwater 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 stormwater 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 stormwater impacts.

Figure 2.9 shows a development that has utilized several lower impact site design techniques in its overall layout and design.

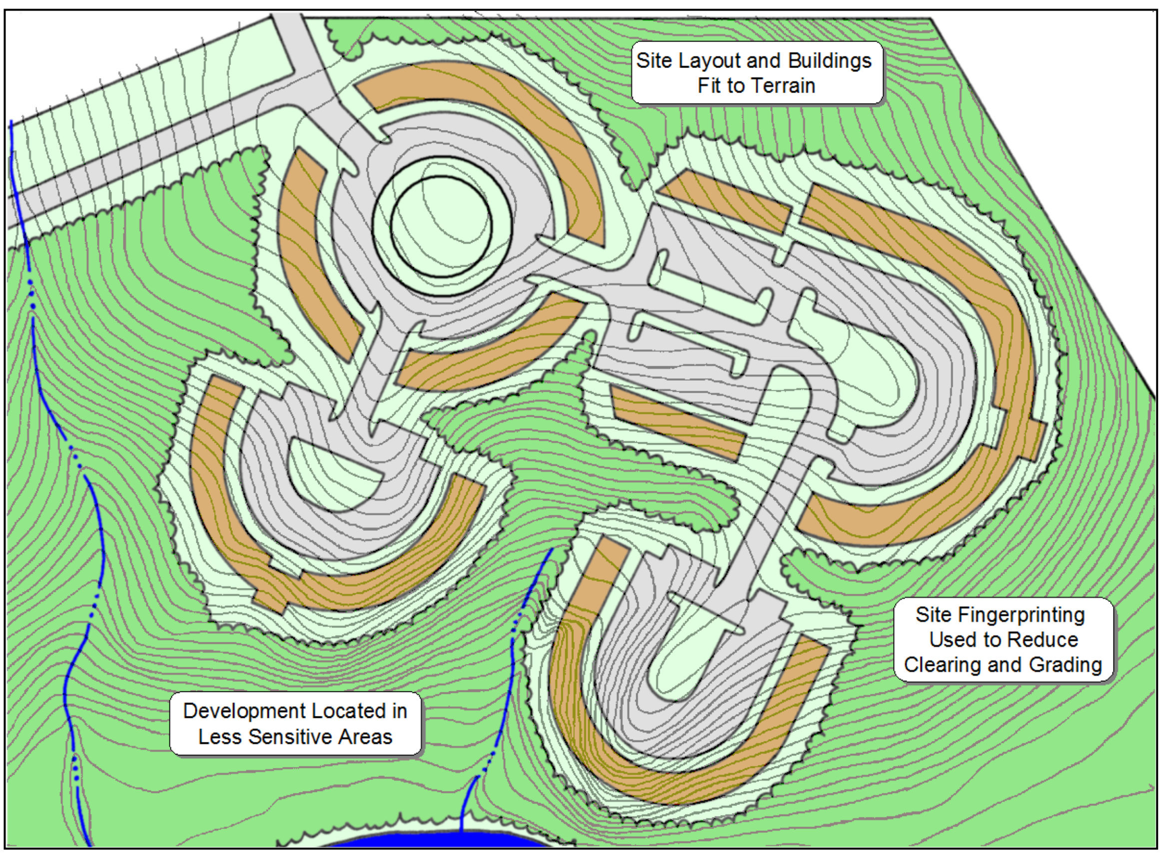


Figure 2.9 Development Design Utilizing Several Lower Impact Site Design Techniques

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 2.10 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 2.11. 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 2.12). 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.

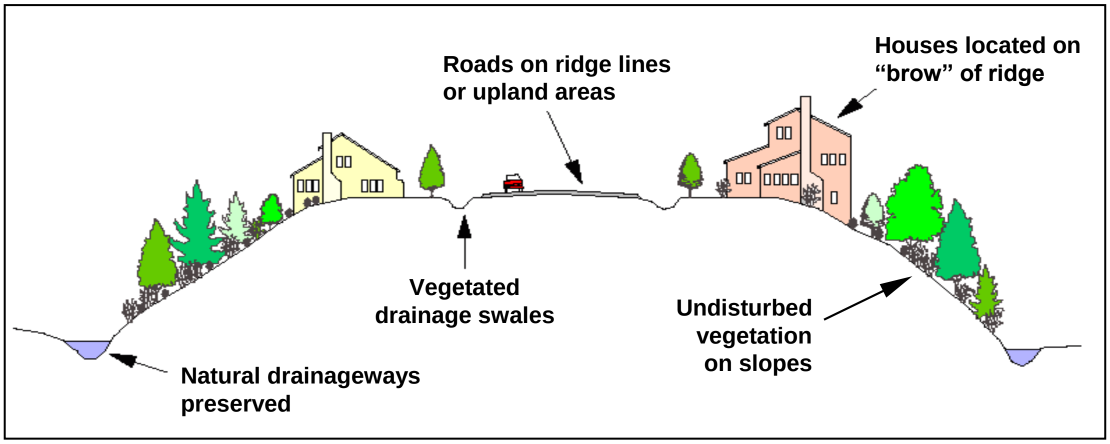


Figure 2.10 Preserving the Natural Topography of the Site
(Adapted from Sykes, 1989)

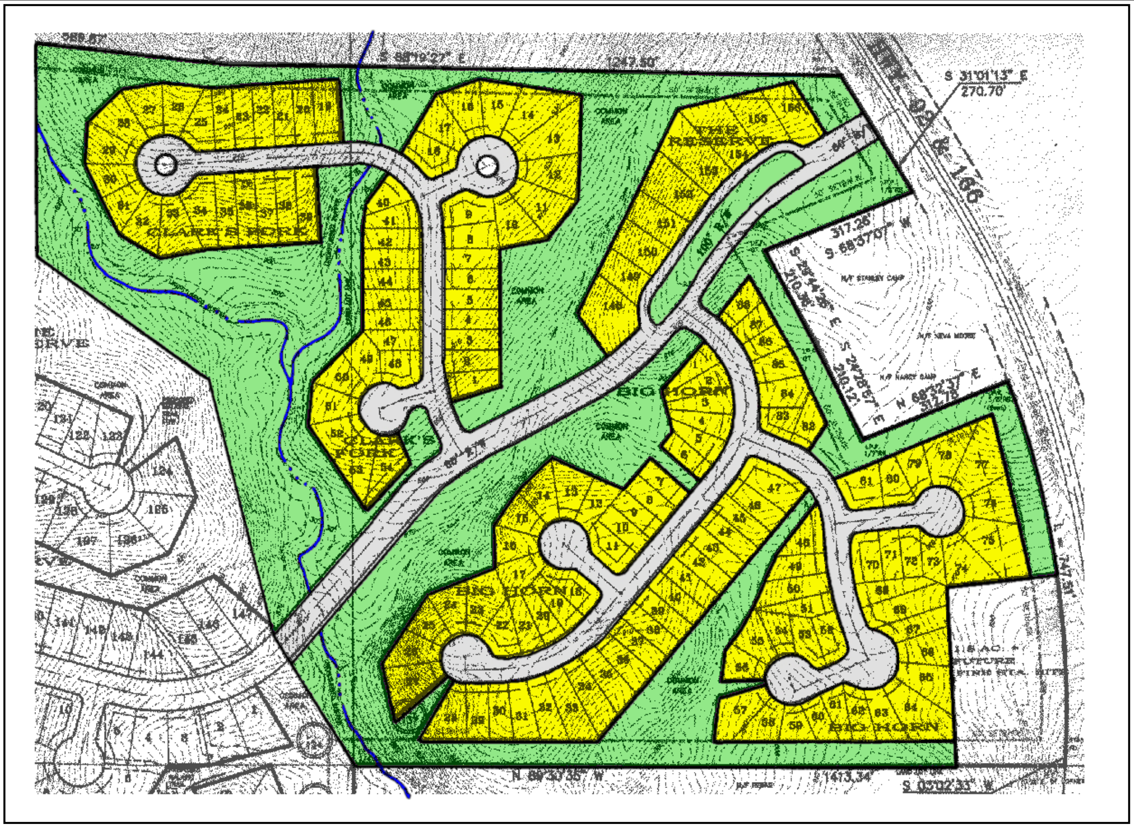


Figure 2.11 Subdivision Design for Hilly or Steep Terrain Utilizes Branching Streets from Collectors that Preserves Natural Drainageways and Stream Corridors

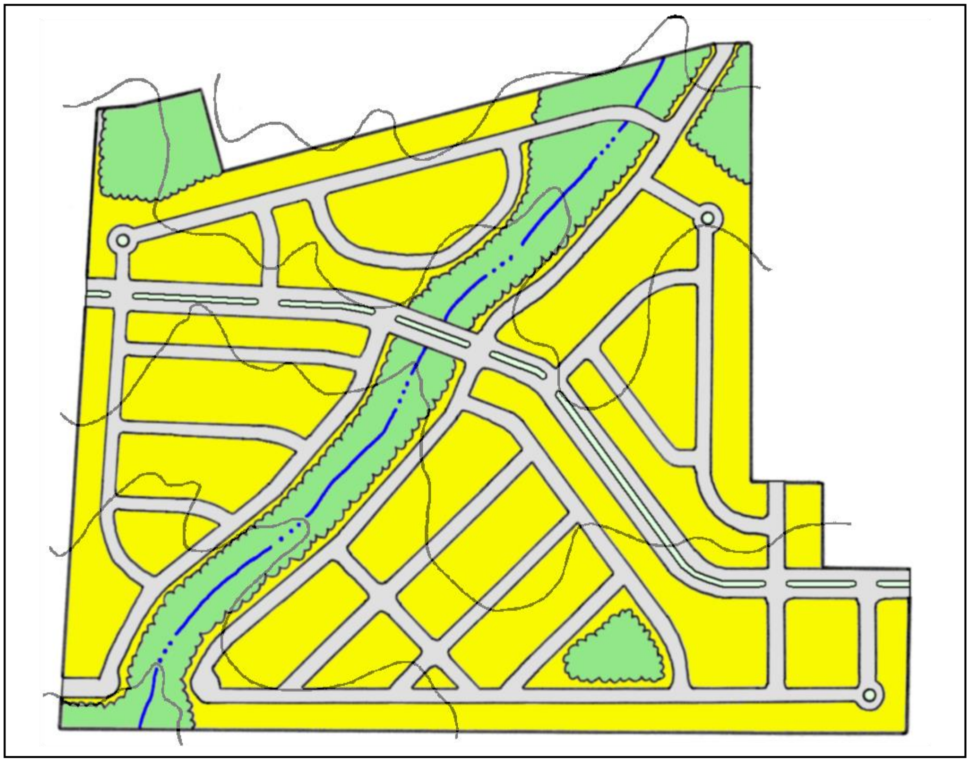


Figure 2.12 A Subdivision Design for Flat Terrain Uses a Fluid Grid Layout that is Interrupted by the Stream Corridor

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 stormwater 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 stormwater runoff infiltration zones. Buildings and impervious surfaces should be located in areas with less permeable soils.

- 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.

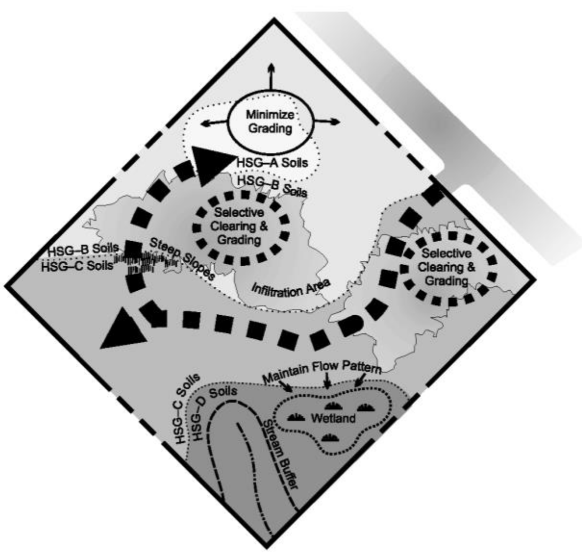


Figure 2.13 Guiding Development to Less Sensitive Areas of a Site
(Source: Prince George's County, MD, 1999)

Figure 2.13 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 2.14 and 2.15.
- Fitting the site design to the terrain.
- Using special procedures and equipment which reduce land disturbance.

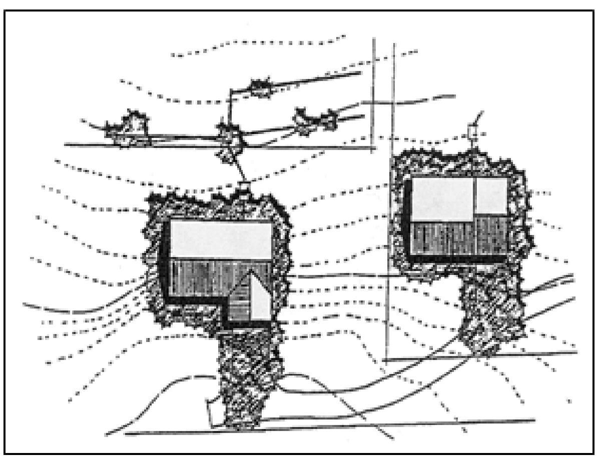


Figure 2.14 Establishing Limits of Clearing
(Source: DDNREC, 1997)

Figure 2.15 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, stormwater pollution, construction costs, and the need for grading and landscaping, while providing for the conservation of natural areas. Figures 2.16 and 2.17 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 stormwater 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.

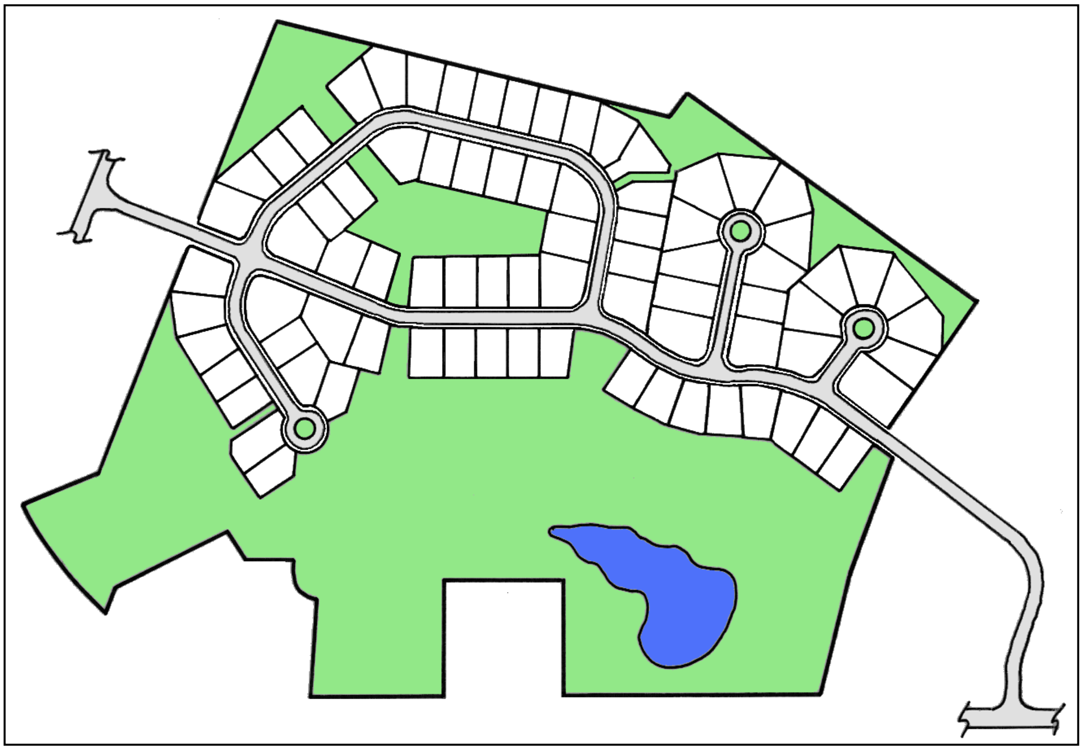


Figure 2.16 Open Space Subdivision Site Design Example

Figure 2.17 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 stormwater 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 stormwater 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.

2.2.3 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 stormwater management. Increased impervious cover means increased stormwater 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 stormwater runoff and associated pollutants that are generated. It can also reduce the size and cost of necessary infrastructure for stormwater 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 Stormwater Islands

Figure 2.18 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 2.18 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 2.19 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.

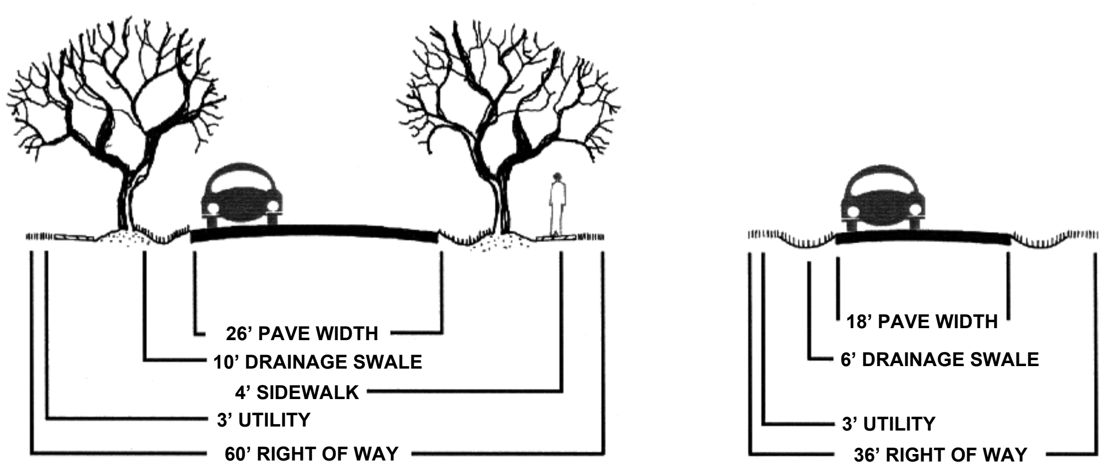


Figure 2.19 Potential Design Options for Narrower Roadway Widths
(Source: VPISU, 2000)

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 2.20 shows the reduction in impervious footprint by using a taller building design.

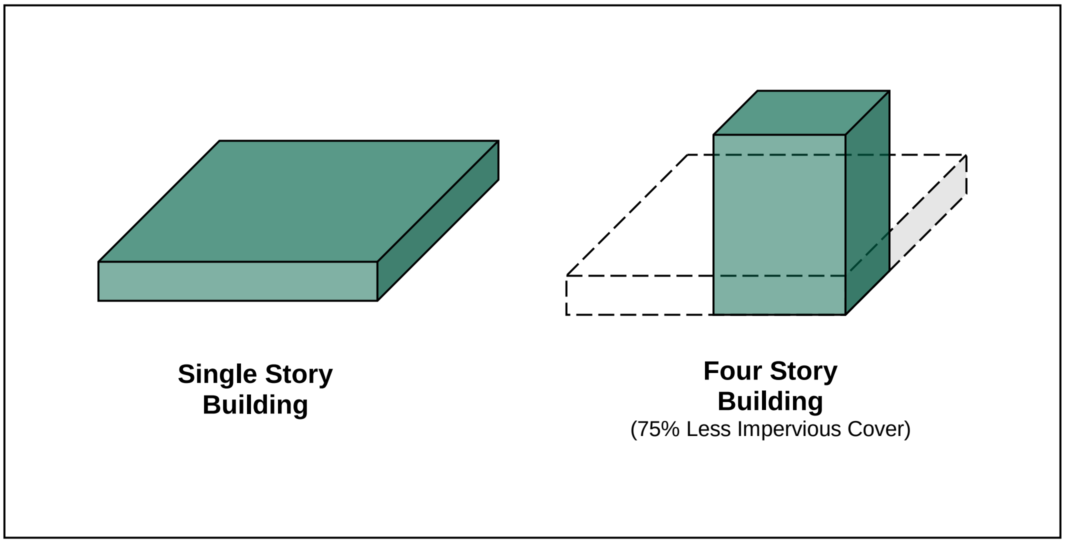


Figure 2.20 Building Up Rather Than Out Can Reduce the Amount of Impervious Cover

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 2.2 provides examples of conventional parking requirements and compares them to average parking demand.

Table 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 2.21 shows a parking deck used for a commercial development.

Figure 2.21 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 2.22 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 the *Site Development Controls Technical Manual* on (Modular Porous Paver Systems) and (Porous Concrete). These surfaces can only be used if the soils allow for adequate infiltration.

Figure 2.22 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 2.23).

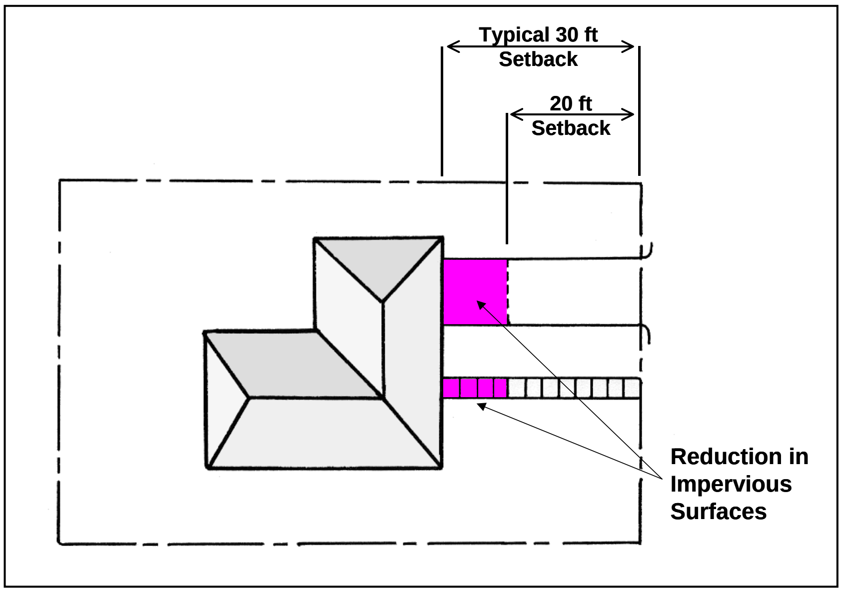


Figure 2.23 Reduced Impervious Cover by Using Smaller Setbacks
(Adapted from: MPCA, 1989)

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 2.24 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 2.25 illustrates various nontraditional lot designs.

Figure 2.24 Examples of Reduced Frontages and Side Yard Setbacks

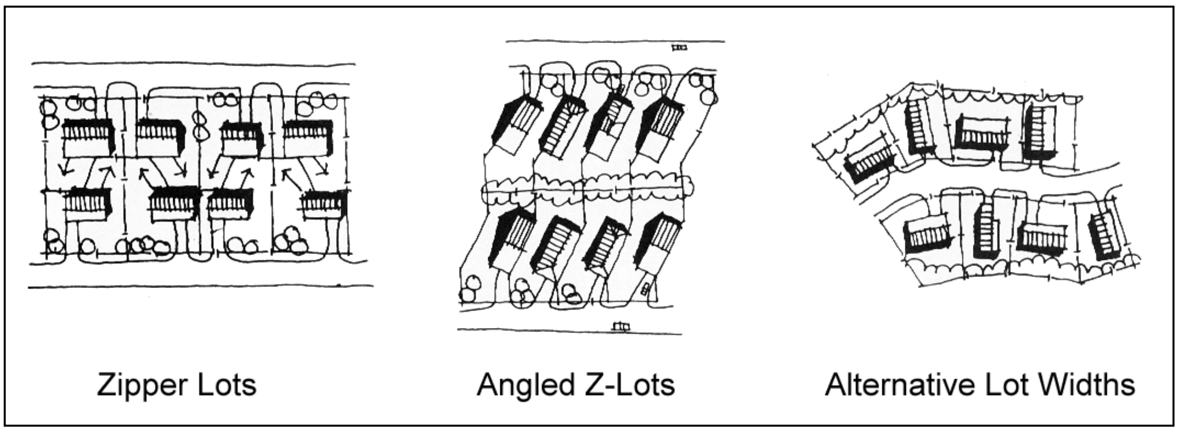


Figure 2.25 Nontraditional Lot Designs
(Source: ULI, 1992)

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

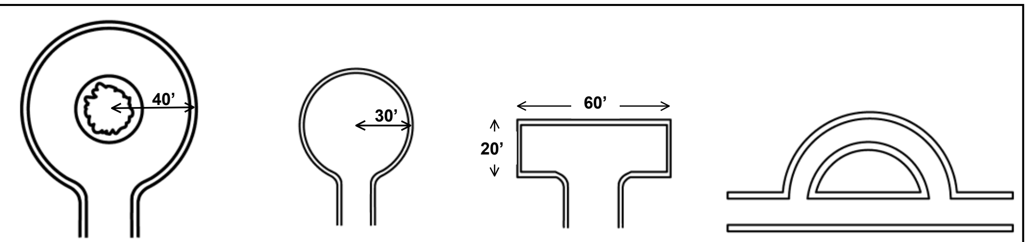
Discussion

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 stormwater 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 2.26).

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.



40 ft cul-de sac with landscaped island | **30 ft radius cul-de-sac** | **60 by 20 ft T-shaped turnaround** | **Loop road**

Figure 2.26 Four Turnaround Options for Residential Streets
(Source: Schueler, 1995)

integrated Site Design Practice #16:

Create Parking Lot Stormwater "Islands"

Reduction of Impervious Cover

Description: Provide stormwater 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 stormwater management "islands" which reduce the overall impervious cover of the lot as well as provide for runoff treatment and control in stormwater 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. Stormwater 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 stormwater 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 the *Site Development Controls Section of the Technical Manual*.

Figure 2.27 Parking Lot Stormwater "Island"

2.2.4 Utilization of Natural Features for Stormwater Management

Traditional stormwater drainage design tends to ignore and replace natural drainage patterns and often results in overly efficient hydraulic conveyance systems. Structural stormwater 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 stormwater filtering of pollutants and sediment; recycle nutrients; and maximize on-site storage of stormwater. 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* stormwater 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.

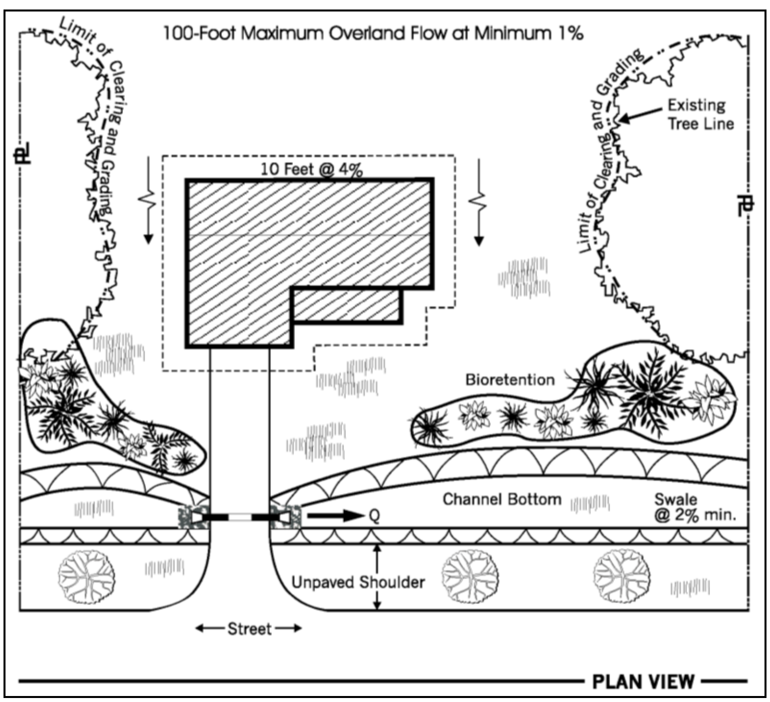


Figure 2.28 Residential Site Design Using Natural Features for Stormwater Management
(Source: Prince George's County, MD, 1999)

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

Utilization of Natural Features for Stormwater Management

Description: Undisturbed natural areas such as forested conservation areas and stream buffers can be used to treat and control stormwater 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 stormwater runoff ▪ Natural depressions can provide inexpensive storage and detention of stormwater flows ▪ A stormwater 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 stormwater 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 2.29. 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 under *Site Development Controls Technical Manual* with a similar goal.

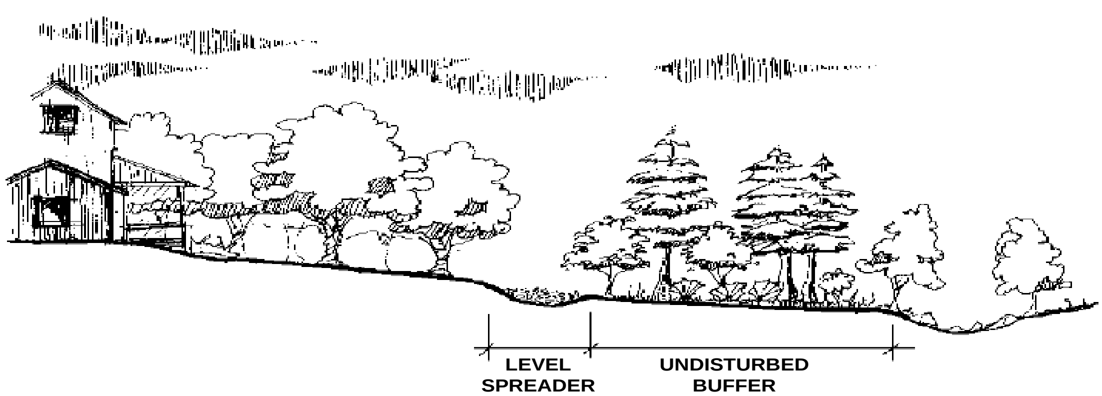


Figure 2.29 Use of a Level Spreader with a Riparian Buffer
(Adapted from NCDENR, 1998)

integrated Site Design Practice #18:

Use Natural Drainageways Instead of Storm Sewers

Utilization of Natural Features for Stormwater 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 stormwater 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 stormwater 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 stormwater flows to their natural outlets, particularly for low-density development and residential subdivisions.

The use of natural open channels (see Figure 2.30) allows for more storage of stormwater flows on-site, lower stormwater peak flows, a reduction in erosive runoff velocities, infiltration of a portion of the runoff volume, and the capture and treatment of stormwater 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.

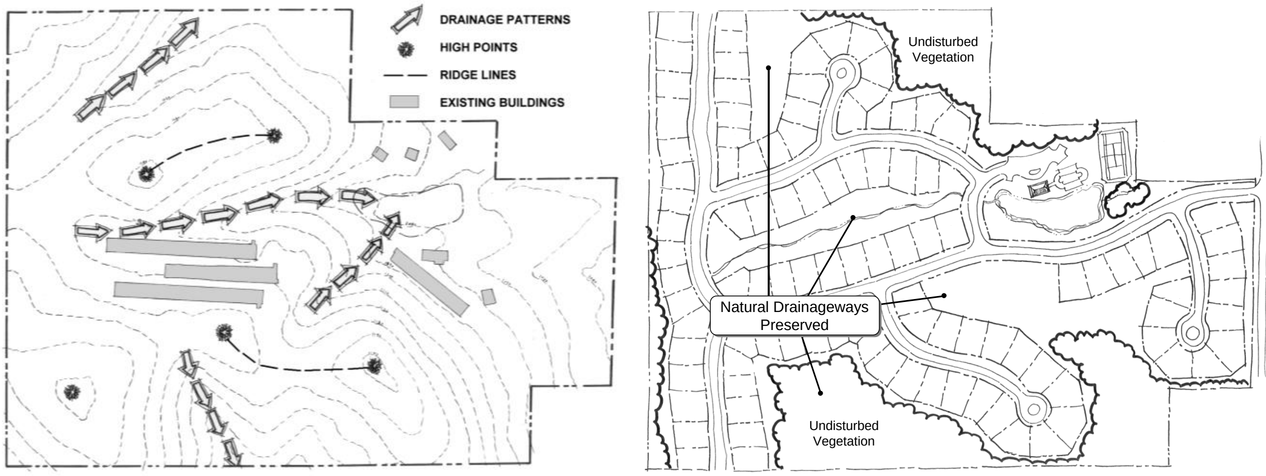


Figure 2.30 Example of a Subdivision Using Natural Drainageways for Stormwater Conveyance and Management

integrated Site Design Practice #19:

Use Vegetated Swales Instead of Curb and Gutter

Utilization of Natural Features for Stormwater 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 stormwater 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 stormwater ▪ A stormwater 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 stormwater runoff

Discussion

Curb and gutter and storm drain systems allow for quicker transport of stormwater 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 stormwater that is often polluted from vehicle emissions, pet waste, lawn runoff and litter.

Open vegetated channels along a roadway (see Figure 2.31) 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 stormwater 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 the *Site Development Controls Section of the Technical Manual*.

Figure 2.31 Using Vegetated Swales Instead of Curb and Gutter

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

Utilization of Natural Features for Stormwater 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 stormwater 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 stormwater runoff ▪ A stormwater 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

Stormwater 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 stormwater 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 2.32)
- 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 the *Site Development Controls Section of the Technical Manual.*

for more design information and specifications on filter strips and vegetated channels.

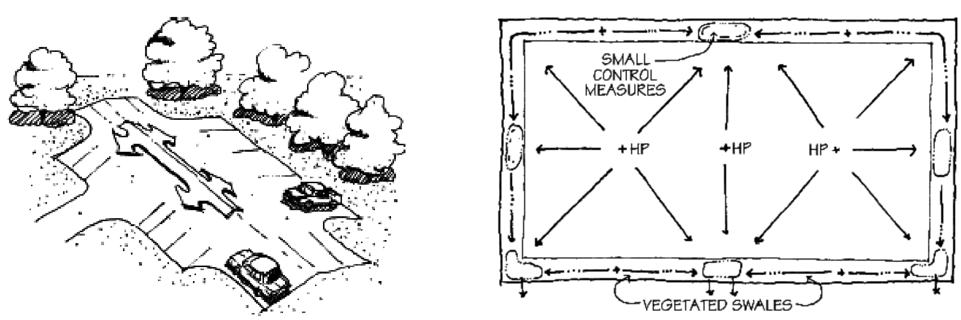


Figure 2.32 Design Paved Surfaces to Disperse Flow to Vegetated Areas
Source: NCDENR, 1998

2.3 *integrated* Site Design Examples

2.3.1 Residential Subdivision Example 1

A typical residential subdivision design on a parcel is shown in Figure 2.33 (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 stormwater flows to the storm sewer system. No provision for non-structural stormwater treatment is provided on the subdivision site.

A residential subdivision employing *integrated* Site Design Practices is presented in Figure 2.33 (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 stormwater 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 stormwater 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.

2.3.2 Residential Subdivision Example 2

Another typical residential subdivision design is shown in Figure 2.34 (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 flood mitigation storm 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 2.34 (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.

2.3.3 Commercial Development Example

Figure 2.35 (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. Stormwater 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 2.35 (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 stormwater 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.

2.3.4 Office Park Example

An office park with a conventional design is shown in Figure 2.36 (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 2.36 (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.

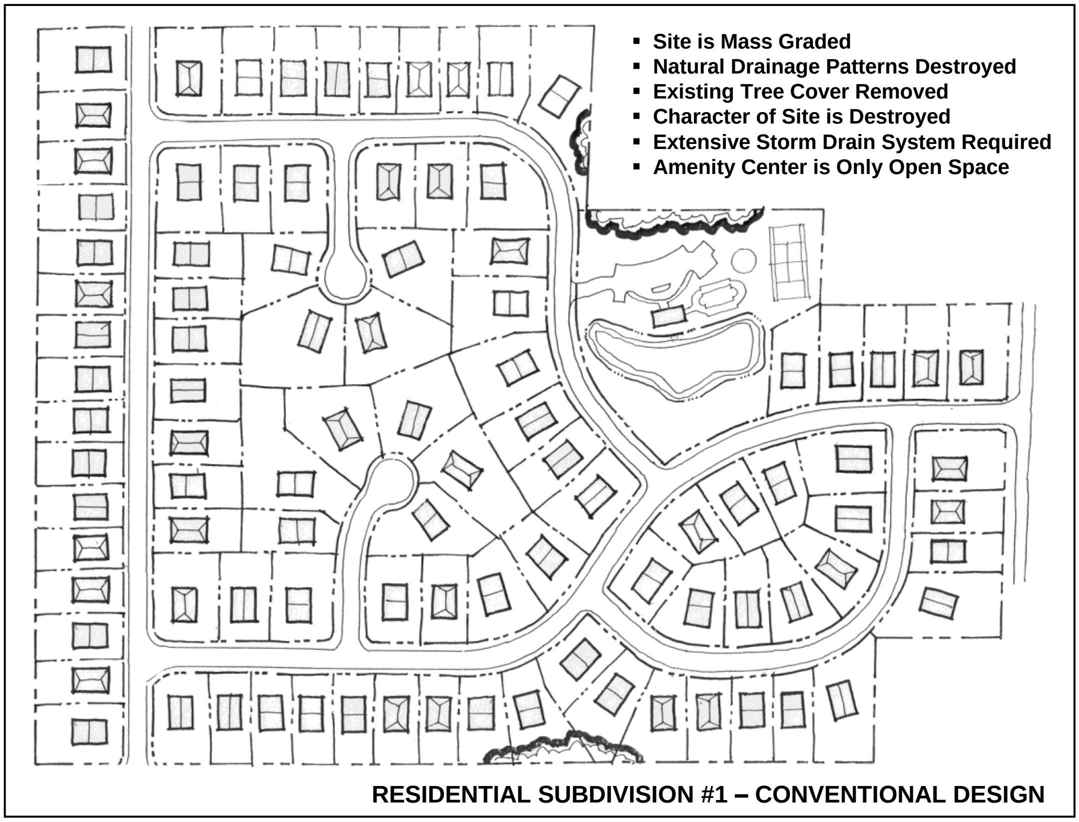


Figure 2.33 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).

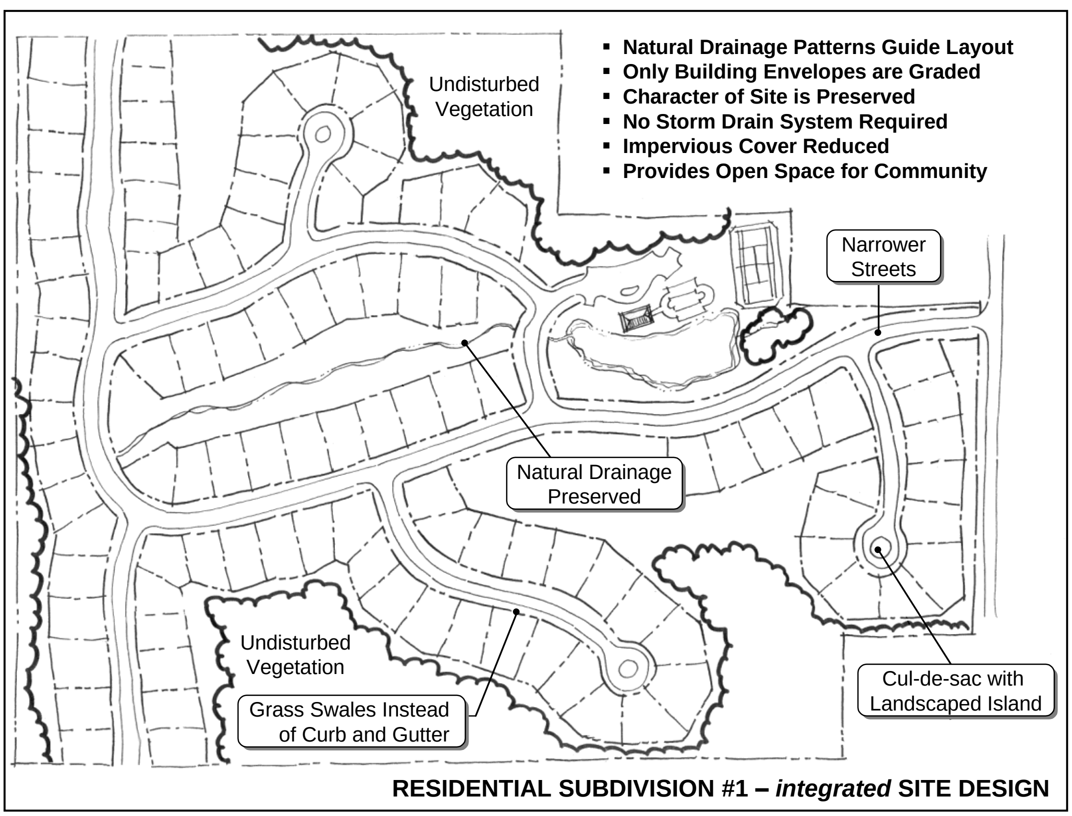
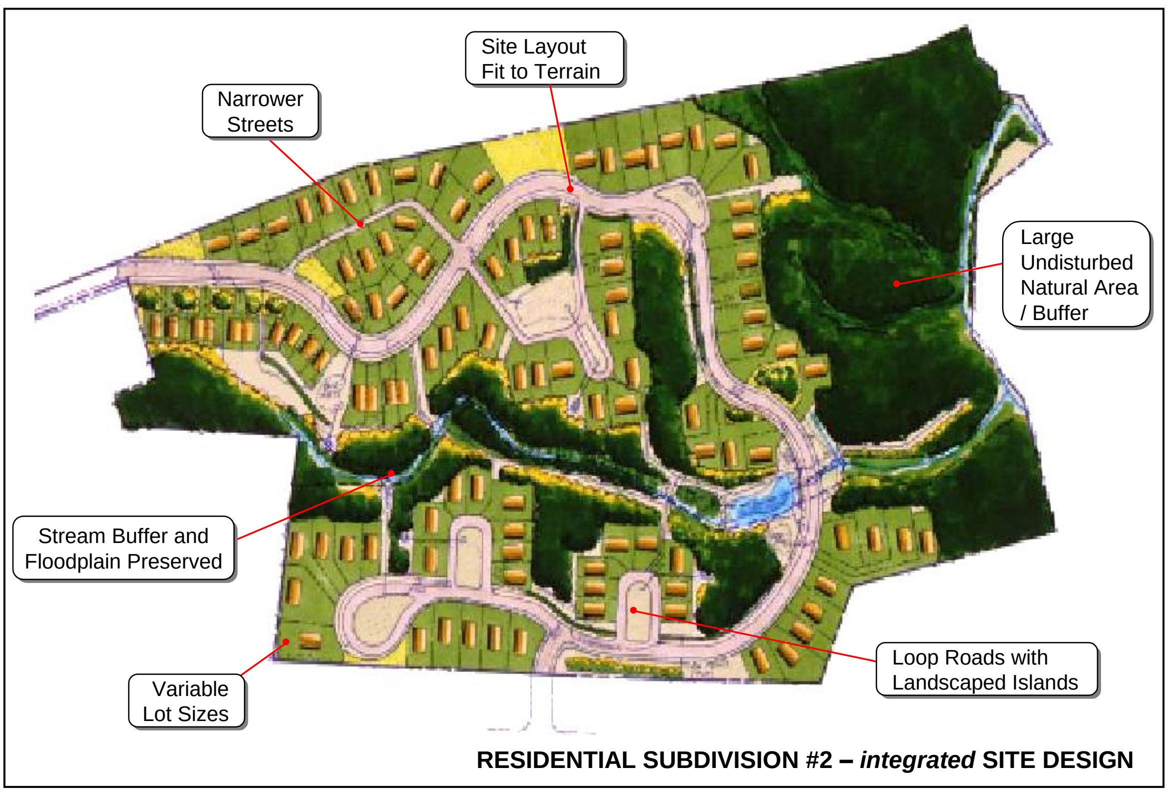


Figure 2.34 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).



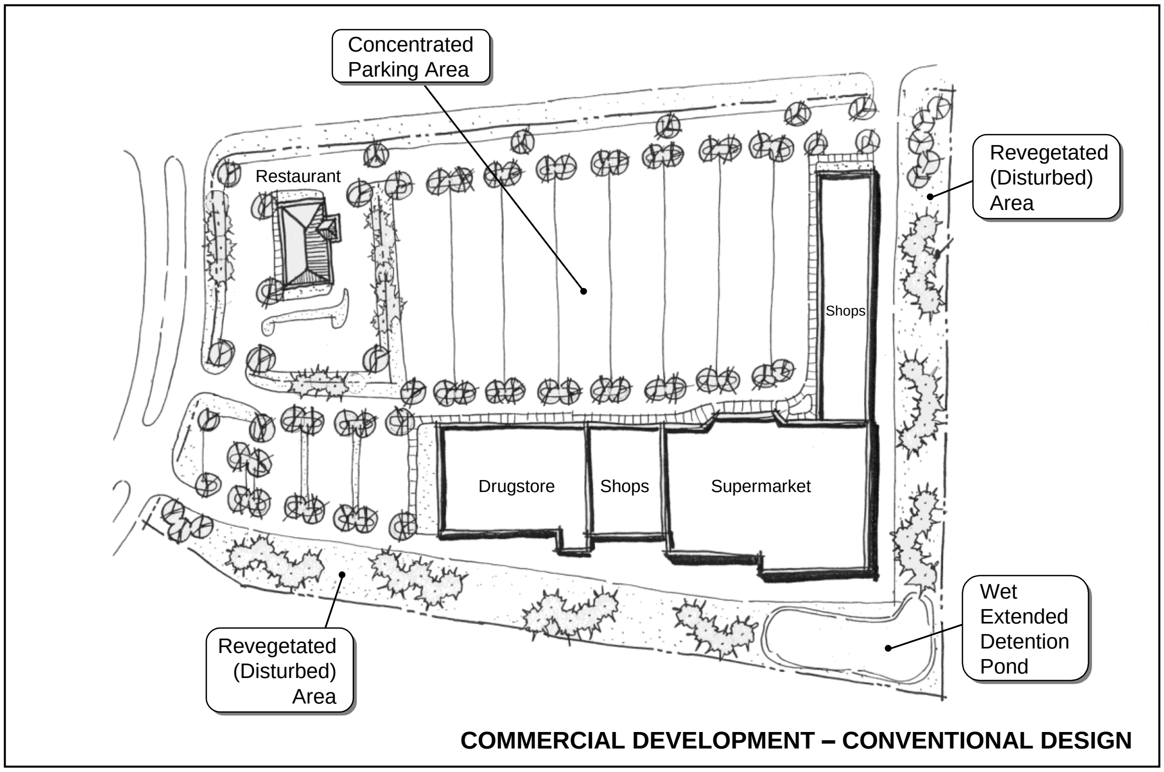
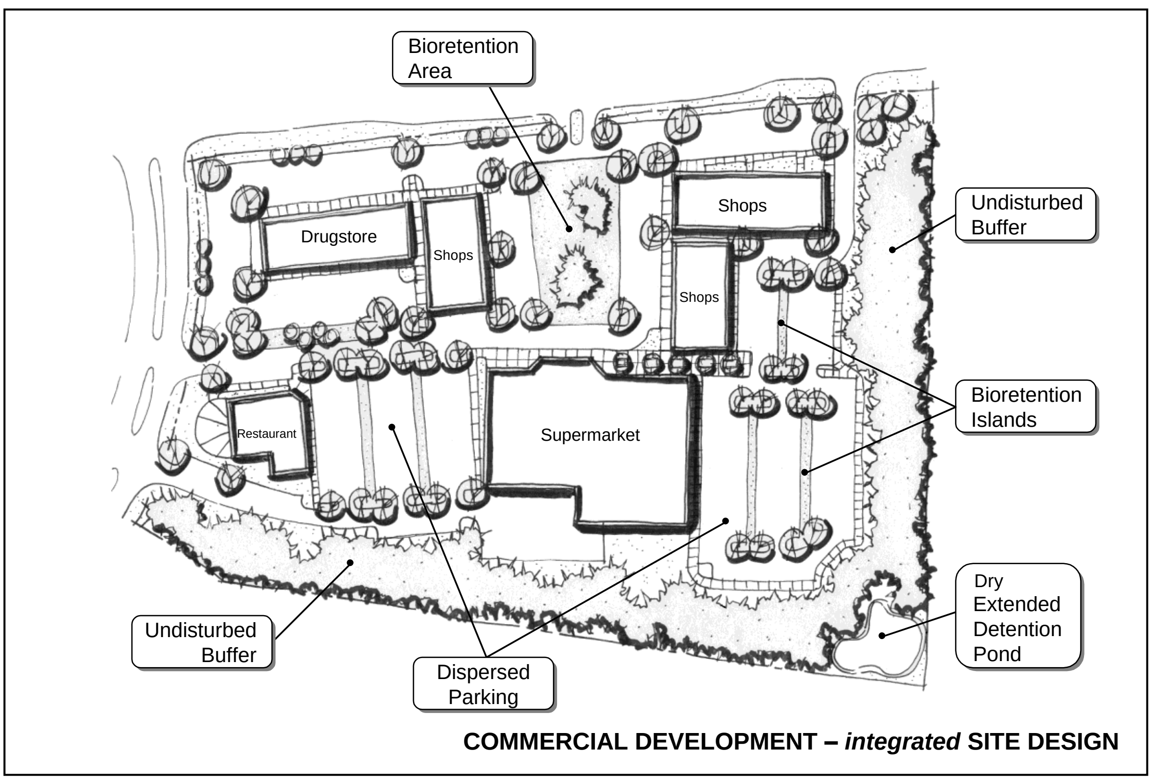


Figure 2.35 Comparison of a Traditional Commercial Development (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).



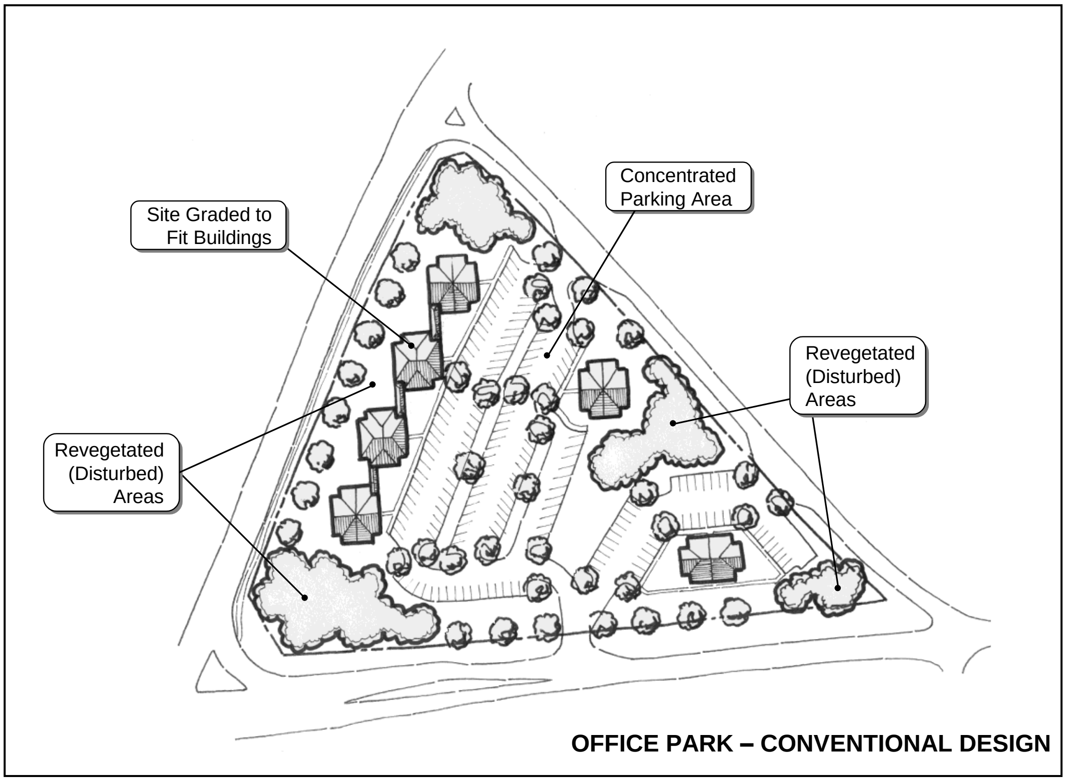
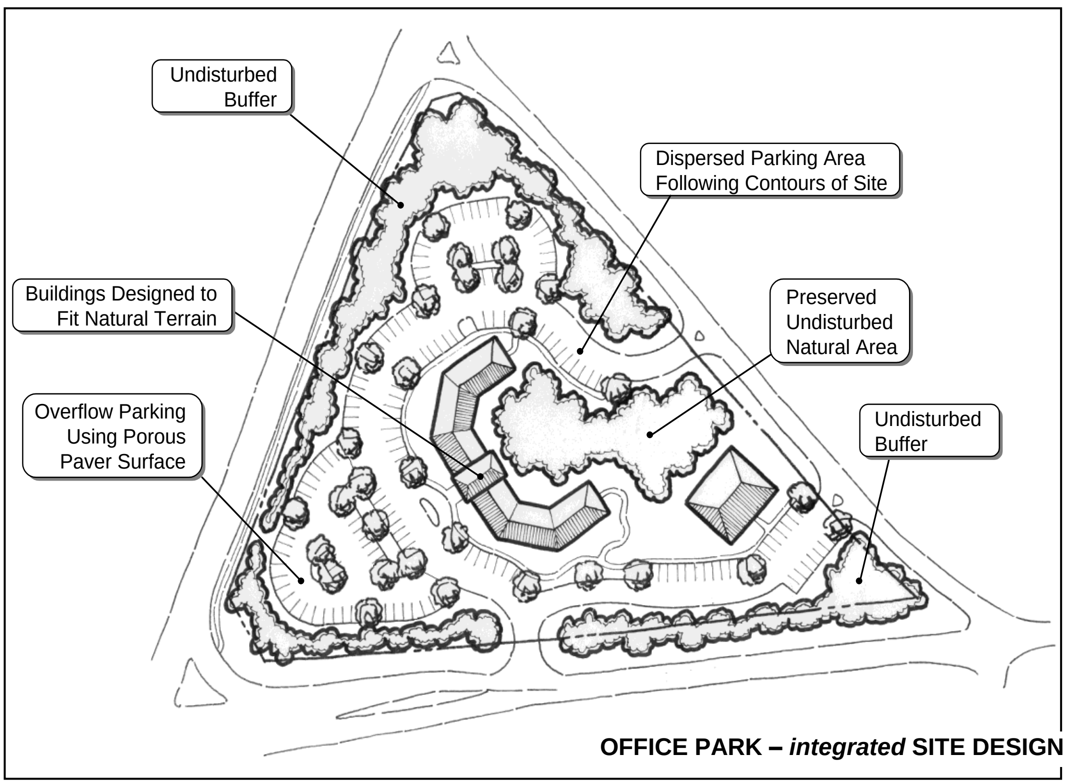


Figure 2.36 Comparison of a Traditional Office Park Design (above) with an Innovative Site Plan Developed Using *integrated* Site Design Practices (below).



2.4 *integrated* Site Design Credits

2.4.1 Introduction

Non-structural stormwater control practices are increasingly recognized as a critical feature in every site design. As such, a set of stormwater "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 stormwater 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.

Refer to *Section 3.2.2 of the Criteria Manual* for more detail on the credits allowed for *integrated* Site Design Practices and the incentives allowed by the local municipality.

Revisions to the iSWM™ Technical Manual
September 2014

In September 2014, revisions to the iSWM™ Technical Manual were made as follows:

Hydrology Technical Manual
- Page HO-46: Clarified calculations in the example problem.

Hydraulics Technical Manual
- Page HA-55: Corrected portion of Equation 1.34 from "A_o-A_i" to "A_o+A_i" based on HEC-22 Manual reference.
- Page HA-96: Rational Method stated as an option for generating inflow hydrographs based on standard regional practice.

Site Development Controls Technical Manual
- Pages SD-25 to SD-26: Updated the method for calculating the pollutant removal rate for controls in series and the corresponding example.
- Pages SD-34 to SD-35: Details added to the bioretention filter media, mulch, and filter fabric requirements.

Construction Controls Technical Manual
- Pages CC-100 to CC-105: Updates to Section 3.6 Organic Filter Tubes, including changes to minimum tube diameter.

Other minor revisions such as corrections of typographical errors or formatting have been made to the iSWM Technical Manual but are not identified.

Water Quality:

1.0 Water Quality Protection Volume and Peak Flow
2.0 Construction SWPPP Guidelines and Form

Table of Contents

List of Tables

1.0 Water Quality Protection Volume and Peak Flow

1.1 Introduction

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 stormwater runoff from larger events, it is possible to effectively mitigate the water quality impacts from a developed area.

1.2 Water Quality Protection Volume Calculation

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 stormwater 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 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 2.2 of the Hydraulics Technical Manual*.

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:

$$\mathbf{R_v = 0.05 + 0.009(I)} \qquad \textbf{(1.1)}$$

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:

$$\mathbf{WQ_v = \frac{1.5\, R_v\, A}{12}} \qquad \textbf{(1.2)}$$

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 (Q_{wv}) using the following formula:

$$\mathbf{Q_{wv} = 1.5(R_v)} \qquad \textbf{(1.3)}$$

where:

Q_{wv} = water quality protection volume (inches)

<u>Determining the Water Quality Protection Volume (WQ_v)</u>

- *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 1.3*.
- *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 1.4*.
- *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 stormwater pond or wetland system (as described in *Section 1.0 of the Site Development Controls Technical Manual*). 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 stormwater 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 stormwater 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 stormwater 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.

1.3 Water Quality Protection Volume Reduction Methods

A set of stormwater "volume reduction methods" is presented to provide developers and site designers an incentive to implement site designs that can reduce the volume of stormwater 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 stormwater 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 1.1. 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 1.1 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	Stormwater 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 stormwater 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 stormwater 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:
$\mathbf{WQ_v = (1.5)(0.38)(38)/12 = 1.81}$ **ac-ft**

With reduction:
$\mathbf{WQ_v = (1.5)(0.38)(31)/12 = 1.47}$ **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 stormwater 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:
$\mathbf{WQ_v = (1.5)(0.38)(38)/12 = 1.81}$ **ac-ft**

With reduction:
$\mathbf{WQ_v = (1.5)(0.38)(33)/12 = 1.57}$ **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 stormwater 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:
$\mathbf{WQ_v = (1.5)(0.62)(3)/12 = 0.23}$ **ac-ft**

With reduction:
$\mathbf{WQ_v = (1.5)(0.62)(2.5)/12 = 0.19}$ **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 stormwater 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)

1.4 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 1.8 of the Hydrology Technical Manual*, a corresponding Curve Number (CN) is computed utilizing the following equation:

$$\mathbf{CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\, Q_{wv}P)^{1/2}]}$$

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 *Section 1.3 of the Hydrology Technical Manual*. Use Type II rainfall distribution for North Central Texas.

(Step 4) Read initial abstraction (I_a) from *Table 1.11 of the Hydrology Technical Manual*, compute I_a/P

(Step 5) Read the unit peak discharge (q_u) from *Figure 1.10 of the Hydrology Technical Manual*, 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 (mi²)

Q_{wv} = Water Quality Protection Volume, in inches ($1.5R_v$)

Example Problem

Using the data and information from the example problem in *Section 1.3.8 of the Hydrology Technical Manual* calculates the water quality volume and the water quality peak flow.

Calculate water quality protection volume (Q_{wv})

Compute volumetric runoff coefficient, R_v

$R_V = 0.05 + (0.009)(I) = 0.05 + (0.009)(18/50 \text{ x } 100\%) = 0.37$

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:

$$CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25\ Q_{wv}\ P)^{½}]$$
$$CN = 1000/[10 + 5*1.5 + 10*0.56 - 10(0.56^2 + 1.25*0.56*1.5)^{½}]$$
$$= 88$$

$t_c = 0.35$ (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 1.10 of the Hydrology Technical Manual* for $I_a/P = 0.18$ $q_u = 580$ cfs/mi²/in

Compute water quality peak flow:

$Q_{wq} = q_u * A * Q_{wv} = 580 * 50/640 * 0.56 = 25.4$ cfs

2.0 Construction SWPPP Guidelines and Form

2.1 State Requirements

The Texas Commission on Environmental Quality (TCEQ) issues TPDES General Permit No. TXR150000 Relating to Discharges Associated with Construction Activities, otherwise known as the TPDES Construction General Permit. The permit is typically effective for a five year period. At each five year renewal of the permit, the TCEQ reviews the permit requirements and makes changes as they deemed necessary. Changes may be based on the TCEQ's experience during the past five year period or may be in response to new criteria from the Environmental Protection Agency (EPA). The current TPDES Construction General Permit can be found at:
www.tceq.state.tx.us/assets/public/permitting/waterquality/attachments/stormwater/txr150000.pdf

The TPDES Construction General Permit requires a Storm Water Pollution Prevention Plan (SWPPP or SWPPP) for construction activity that:

- Disturbs one acre or more of land; or
- Disturbs less than one acre of land and is part of a common plan of development that disturbs one acre or more of land.

Requirements for the SWPPP are in Part III of the TPDES Construction General Permit. Currently, the TCEQ does not provide guidance on preparing a SWPPP. EPA provides SWPPP guidance, templates and examples at: http://cfpub.epa.gov/npdes/stormwater/swppp.cfm#guide

2.2 Disclaimer

The Construction SWPPP Guidelines presented in this section of the iSWM Technical Manual were developed for the 2003 TPDES Construction General Permit which expired in 2008. Some of the content in these guidelines does not reflect SWPPP requirements in the current permit. In addition, the EPA published a Final Rule for Effluent Limitations Guidelines and Standards for the Construction and Development Point Source Category on December 1, 2009. This rule will significantly alter the SWPPP requirements for construction activities as it is phased into effect over the next few years. The North Central Texas Council of Governments is in the process of reviewing the new rule and determining how these guidelines should be revised.

The basic methodology of the SWPPP design and implementation presented in these guidelines is still sound. Users of the iSWM Technical Manual may use the guidelines to develop a SWPPP. However, they must separately ensure compliance with the SWPPP requirements in the current TPDES Construction General Permit.

The North Central Texas Council of Governments or local municipalities make no claim regarding the conformity of the information presented in these guidelines with state or federal regulations. Use of the information and/or SWPPP Narrative form does not in any way guarantee that the user will be in compliance with the TPDES Construction General Permit. The North Central Texas Council of Governments or local municipalities assume no liability for prosecution arising out of the use of the information and/or SWPPP Narrative form.

For projects requiring permit coverage, the Storm Water Pollution Prevention Plan should be checked thoroughly for compliance with current TPDES Construction General Permit.

2.3 Overview

The Storm Water Pollution Prevention Plan (SWPPP) is the primary tool for reducing erosion and preventing sediment loss from a construction site and the developed property once the building is placed in service. It consists of a narrative and drawings of the existing conditions and control methods to be employed during the land disturbance and construction process. Storm Water Pollution Prevention Plans shall be prepared in accordance with good engineering practices and prepared by someone with a background in hydrology or hydraulics and familiar with sediment and erosion control. It is recommended that a Certified Professional in Erosion and Sediment Control (CPESC) or qualified engineer or landscape architect prepare the SWPPP. Note that there are a few MS4 municipalities in North Central Texas that require the SWPPP to be prepared by a registered Professional Engineer.

These guidelines incorporate the design elements of the iSWM Criteria Manual along with the general provisions of the TPDES Construction General Permit. The iSWM Technical Manual includes a form for use as a model in developing the narrative portion of a SWPPP.

Note: A few local governments in North Central Texas require the Construction SWPPP to meet a numeric design guideline (site rating) for erosion minimization and sediment retention on construction sites. The Construction Controls section of the iSWM Technical Manual contains methodology for the Site Rating Calculation.

2.4 Elements of a Construction SWPPP

An adequate Construction SWPPP includes a narrative and drawings. The narrative is a written statement to explain and justify the pollution prevention decisions made for a particular project. The narrative contains concise information about existing site conditions, construction schedules, and other pertinent items that may not be contained on the drawings. The drawings and notes describe where and when the various controls should be installed, the performance the controls are expected to achieve, and actions to be taken if the performance goals are not achieved.

2.4.1 Narrative

<u>Project description</u>: Describe the nature and purpose of the construction activity. Include the size (in acres) of the entire property and the area to be disturbed by project construction, including off-site material storage areas, overburden and stockpiles of dirt, and borrow areas.

<u>Existing topography and natural drainage features</u>: Describe the existing topography, drainage patterns, and natural drainage features on the site including channels, creeks, watercourses, etc. Provide name (if available) of creeks, streams, etc. and protection measures such as buffers. Provide the name of receiving waters.

<u>Existing storm sewer system</u>: Describe existing onsite storm sewer systems including location of inlets and outfalls, pipe sizes, etc. Provide a description of the downstream drainage facilities leading from the site to the receiving body of water.

<u>Soils</u>: Describe the soil on the site, giving such information as the soil type(s) and erodibility (low, medium, high or an index value from the county Natural Resource Conservation Service (NRCS) soil survey. Identify any unique site characteristics that may not be shown on the soil survey based upon a field visit.

<u>Ground cover</u>: Label existing vegetation on the drawing. Such features as tree clusters, grassy areas, and unique or sensitive vegetation should be shown.

<u>Critical areas</u>: Describe the location, size, and characteristics of any wetlands, streams, or lakes that are within the site, and/or will receive discharges from disturbed areas of the project, and protection

measures. Note areas with high erosion potential including steep slopes and flood hazard areas. Describe special requirements for working near or within any of these areas.

Potential pollutants: Describe potential pollutants, including construction and waste materials, chemicals, paints, solvents, fuels, etc expected to be stored on-site and controls to minimize pollutant discharges.

Construction support activities: Describe any on-site or off-site asphalt or concrete batch plants, equipment staging, repair, or refueling areas, and material storage areas providing sole and direct support to the construction project and controls that will be implemented to minimize pollutant discharges.

Construction schedule: Describe the intended schedule or sequence of major activities that will disturb soils for major portions of the site. Describe the general timing or sequence for implementation (and removal) of BMPs that will be used to minimize pollution in runoff. Describe the average monthly rainfall and rainfall intensity for required design storm events during the anticipated schedule.

Engineering calculations: Attach any calculations made for the design of such items as sediment basins and temporary swales, dikes, and channels. For sediment basins, engineering calculations must bear the signature and stamp of an engineer licensed in the state of Texas.

Elements of a Construction SWPPP: Describe how the Construction SWPPP addresses each of the following elements of a Construction SWPPP. Include the type and location of controls used to satisfy the required element and the general timing or sequence for implementation. If one or more of the elements are not applicable to a project, provide a written justification for why the particular element(s) is (are) not necessary. Refer to *Section 4.3 of the Criteria Manual* for additional details on the elements.

1. Limits of Disturbance - Description of the areas including natural drainage features, trees and other vegetation, and appropriate buffers that are to be preserved within the construction area and the measures to be implemented to ensure protection.
2. Stabilization to Prevent Soil Erosion - Description of the temporary and final stabilization practices for disturbed areas of the site, including a schedule of when the practices will be implemented.
3. Slope Protection - Description of the practices used to protect slopes and divert flows away from exposed soils or disturbed areas.
4. Sediment Barriers and Perimeter Controls - Description of the practices to lessen the off-site transport of sediment and to reduce generation of dust. Sediment basins are required, where feasible, for common drainage locations that serve an area with ten or more acres disturbed at one time.
5. Velocity Dissipation and Channel Protection - Description of velocity dissipation devices used at discharge locations and channel stabilization measures to provide non-erosive flows.
6. Construction Access Controls - Description of measures to minimize the off-site tracking of sediment by vehicles.
7. Storm Drain Inlet Protection - Description of inlet protection measures to prevent sediment from entering the storm drain system.
8. Dewatering Controls - Description of controls to prevent the off-site transport of suspended sediments and other pollutants in discharges from dewatering operations.
9. Material and Waste Controls - Description of controls to reduce pollutants and spill prevention and response procedures associated with construction and waste materials. Description of controls and measures that will be implemented to minimize pollutants in any discharges associated with industrial activity other than construction (i.e., dedicated asphalt or concrete plants).
10. Construction Phasing and Project Management Description of considerations given to project phasing in order to reduce the amount of soil exposed at one time.

Permanent Storm Water Controls: Describe any measures that will be installed during the construction process to control pollutants in discharges from the site after construction operations have been completed.

Copy of the TPDES Construction General Permit: Include a copy of TPDES Construction General Permit TXR 150000 with the SWPPP. Also include a copy of the Notice(s) of Intent for large construction activities or the Construction Site Notice(s) for large and small construction activities as part of the SWPPP.

2.4.2 Drawings

Vicinity map: Provide a map providing the general location of the site in relation to the surrounding area and roads.

Site map: Provide one or more plan sheets drawn to scale showing the following features. The site map requirements may be met using multiple plan sheets for ease of legibility.

- Location of property boundaries.
- The direction of north in relation to the site and scale of the drawing.
- Existing structures and roads, if present.
- Identification of and approximate boundaries for existing soil types.
- Existing topography (maximum 2' contour interval) with at least two contour elevations labeled on each plan sheet.
- Limits of on-site surface waters and adjacent critical areas (including wetlands), their buffers, and FEMA base flood boundaries.
- Limits of drainage subbasins and the direction of flow for the different drainage areas (before and after major grading activities).
- Drainage features such as drainage systems, channels, and natural watercourses.
- Surface waters, including wetlands, adjacent or in close proximity to the site.
- Locations where storm water runoff discharges from the site directly to a surface water body.
- Boundaries of areas where soil disturbance will occur (clearing and grading limits).
- Final grade contours (maximum 2' contour interval) with at least two contour elevations labeled on each plan sheet and approximate slopes indicated.
- Existing vegetation and the vegetation that is to be preserved.
- Limits and time frame of construction phases.
- Approximate slope and cut and fill slopes indicating top and bottom of slope catch lines for grades exceeding 5%.
- Locations of on-site or off-site waste, chemical, fuel, and equipment storage, fueling, or maintenance areas.
- Locations of on-site or off-site material, waste, borrow or fill areas.
- Locations of any asphalt or concrete batch plants providing sole and direct support to the construction site.

Conveyance systems: Show on the site map the following temporary and permanent conveyance features.

- Channels associated with erosion and sediment control and storm water management.

- Locations of temporary and permanent storm drain pipes.
- Slope, dimensions, and direction of flow in swales, dikes, channels, culverts, and pipes.
- Locations and outlets of any dewatering systems.

Erosion and Sediment Controls: Show on the site map the locations for all of the controls described for the elements in the narrative portion of the SWPPP. The controls must satisfy each of the elements of a Construction SWPPP, unless justification is provided in the narrative for not including one or more of the elements. Show construction details and specifications for the controls. Some examples of controls include (but are not limited to):

- Locations of interceptor swales or diversion dikes and details for bypassing off-site runoff around disturbed areas.
- The location of sediment basin(s) and appropriate details including overall dimensions, storage volume, inflow and release rates, riser barrel/outlet assembly, overflow, embankment, etc.
- Flow depth and velocity for proposed swales and channels and other measures to control flow rates and stabilize channels.
- Locations and details for inlet and outlet protection practices.
- Locations and details for mulch and/or recommended cover of slopes.
- Locations and details for temporary or final vegetation (stabilization practices).
- Locations and details for check dams.
- Locations and details for organic filter berms, silt fence, organic filter tubes, or triangular sediment dikes.
- Locations of passive or active treatment systems.
- The construction entrance location and details.
- Any permanent (post-construction) storm water management controls to be installed during the construction phase.
- Locations of controls for pollutants from construction support activities.
- Location of controls for pollutants other than sediment.

Detailed drawings: Any structural practices used that are not referenced in this manual or other local manuals should be explained and illustrated with detailed drawings.

2.5 SWPPP Review Checklist

A checklist is provided for local municipalities to use in reviewing Storm Water Pollution Prevention Plans for conformance with the provisions of this Manual. The checklist is also useful for SWPPP designers to check the contents of the SWPPP.

Storm Water Pollution Prevention Plan Review Checklist

Project Name____________________________

Location/Address ____

Operator's Name _________________________ Operator's Phone No. __________________

Preparer's Name _________________________ Preparer's Phone No. __________________

Reviewer _______________________________ Date __________________

I. NARRATIVE

_____ Project Title

_____ Operator with Control Over Construction Plans and Specifications

- _____ Company Name and Address
- _____ Name and Phone Number of Operator's Representative
- _____ Name of Preparer and Date Prepared
- _____ Certification Statement in accordance with TPDES Construction General Permit TXR150000
- _____ Signatory Name and Title (printed) and Signature

_____ Operator(s) with Day-to-Day Operational Control Over Activities to Ensure Compliance with SWPPP

- _____ Company Name(s) and Address(es)
- _____ Name and Phone Number of Operator's Representative

_____ Copy of NOI(s) or Site Notice(s) and TPDES Construction General Permit TXR150000

_____ Name of Receiving Water(s)

_____ Name of Municipal Separate Storm Sewer System (MS4) Receiving Discharge (if applicable)

_____ Description of Project/Construction Activity

_____ Total Area of Property

_____ Total Area of Site to be Disturbed by Project Construction

_____ Total Area of Off-site Material Storage Areas, Overburden & Stockpiles of Dirt, and Borrow Areas

_____ Description of Existing Topography and Natural Drainage Features (pre-construction drainage patterns, and natural drainage features including channels, creeks, watercourses, etc.)

_____ Description of Existing Storm Sewer System (existing onsite storm sewer systems including location of inlets and outfalls, pipe sizes, etc.)

_____ Description of Soils (i.e., soil types, erodibility, unique site soil features, etc.)

_____ Description of Existing/Pre-construction Ground Cover (i.e.tree clusters, grassy areas, and unique or sensitive vegetation)

_____ Description of Critical Areas (location, size, and characteristics of any wetlands, streams, or lakes that are adjacent or in close proximity to the site, and/or will receive discharges from disturbed areas of the project; steep slopes and areas with high erosion potential)

_____ Description of Potential Pollutants (i.e.,construction and waste materials, chemicals, paints, solvents, etc expected to be stored on-site)

_____ Construction Support Activities (on-site or off-site asphalt or concrete batch plants, equipment staging, repair, or refueling areas, and material storage areas providing sole and direct support to the construction project and controls that will be implemented to minimize pollutant discharges)

_____ Sequence of Major Construction Activities (intended sequence of major activities that will disturb soils and general timing or sequence for implementation (and removal) of controls)

_____ Description of Permanent/Post-Construction Storm Water Management Controls (measures that will be installed to control pollutants in storm water discharges that will occur after construction is complete and the developed property is placed in service)

_____ Engineering Calculations (review calculations made for the design of such items as sediment basins and temporary swales, dikes, and channels. For sediment basins, engineering calculations must bear the signature and stamp of an engineer licensed in the state of Texas)

Elements of a Construction SWPPP

For each of the following elements, evaluate the description of the measures used to address the element. Evaluate the type and location of controls used to satisfy the required element and the general timing or sequence for implementation. If an element is indicated as not applicable to a project, evaluate the written justification for why it is not necessary.

_____ 1. Limits of Disturbance - Description of the areas including natural drainage features, trees and other vegetation, and appropriate buffers that are to be preserved within the construction area and the measures to be implemented to ensure protection.

_____ 2. Stabilization to Prevent Soil Erosion - Description of the temporary and permanent stabilization practices for disturbed areas of the site, including a schedule of when the practices will be implemented.

_____ 3. Slope Protection - Description of the practices used to protect slopes and divert flows away from exposed soils or disturbed areas.

_____ 4. Sediment Barriers and Perimeter Controls - Description of the practices to lessen the off-site transport of sediment and to reduce generation of dust. Sediment basins are required, where feasible, for common drainage locations that serve an area with ten or more acres disturbed at one time.

_____ 5. Velocity Dissipation and Channel Protection - Description of velocity dissipation devices used at discharge locations and channel stabilization measures to provide non-erosive flows.

_____ 6. Construction Access Controls - Description of measures to minimize the off-site tracking of sediment by vehicles.

_____ 7. Storm Drain Inlet Protection - Description of inlet protection measures to prevent sediment from entering the storm drain system.

_____ 8. Dewatering Controls - Description of controls to prevent the off-site transport of suspended sediments and other pollutants in discharges from dewatering operations.

_____ 9. Material and Waste Controls - Description of controls to reduce pollutants and spill prevention and response procedures associated with construction and waste materials. Description of controls and measures that will be implemented to minimize pollutants in any discharges associated with industrial activity other than construction (i.e., dedicated asphalt or concrete plants).

_____ 10. Construction Phasing and Project Management - Description of considerations given to project phasing in order to reduce the amount of soil exposed at one time.

II. DRAWINGS

_____ Vicinity map (providing the general location of the site in relation to the surrounding area and roads).

Site map: Verify that the plan sheets are drawn to scale and show the following features:

_____ Location of property boundaries

_____ The direction of north in relation to the site and scale of the drawing

_____ Existing structures and roads, if present

_____ Identification of and approximate boundaries for existing soil types

_____ Existing topography (maximum 2' contour interval) with at least two contour elevations labeled on each plan sheet

_____ Limits of on-site surface waters and adjacent critical areas (including wetlands), their buffers, and FEMA base flood boundaries

_____ Limits of drainage subbasins and the direction of flow for the different drainage areas (before and after major grading activities)

_____ Drainage features such as drainage systems, channels, and natural watercourses

_____ Surface waters, including wetlands, adjacent or in close proximity to the site

_____ Locations where storm water runoff discharges from the site directly to a surface water body

_____ Boundaries of areas where soil disturbance will occur (clearing and grading limits)

_____ Final grade contours (maximum 2' contour interval) with at least two contour elevations labeled on each plan sheet and approximate slopes indicated

_____ Existing vegetation and the vegetation that is to be preserved

_____ Limits and time frame of construction phases

_____ Approximate slope and cut and fill slopes indicating top and bottom of slope catch lines for grades exceeding 5%

_____ Locations of on-site or off-site waste, chemical, fuel, and equipment storage, fueling, or maintenance areas

_____ Locations of on-site or off-site material, waste, borrow or fill areas

_____ Locations of any asphalt or concrete batch plants providing sole and direct support to the construction site

Conveyance systems: Show on the site map the following temporary and permanent conveyance features.

_____ Channels associated with erosion and sediment control and storm water management

_____ Locations of temporary and permanent storm drain pipes

_____ Slope, dimensions, and direction of flow in swales, dikes, channels, culverts, and pipes

_____ Locations and outlets of any dewatering systems

Erosion and Sediment Controls: Check the site map for the locations of all the controls described for the elements in the narrative portion of the SWPPP. Check for construction details and specifications for the controls. The controls must satisfy each of the elements of a Construction SWPPP, unless justification was provided in the narrative for not including one or more of the elements. Some examples of controls include (but are not limited to):

_____ Locations of interceptor swales or diversion dikes and details for bypassing off-site runoff around disturbed areas

_____ The location of sediment basin(s) and appropriate details including overall dimensions, storage volume, inflow and release rates, riser barrel/outlet assembly, overflow, embankment, etc.

_____ Flow depth and velocity for proposed swales and channels and BMPs to control flow rates and stabilize channels

_____ Locations and details for inlet and outlet protection practices

_____ Locations and details for mulch and/or recommended cover of slopes

_____ Locations and details for temporary or permanent vegetation (stabilization practices)

_____ Locations and details for check dams

_____ Locations and details for organic filter berms, silt fence, organic filter tubes, or triangular sediment dikes

_____ Locations of passive or active treatment systems

_____ Construction entrance location and details

_____ Any permanent (post-construction) storm water management controls to be installed during the construction phase

_____ Locations of controls for pollutants from construction support activities

_____ Location of controls for pollutants other than sediment

Detailed drawings: Any structural practices used that are not referenced in this manual or other local manuals should be explained and illustrated with detailed drawings.

Corrective Actions Required

____
____
____
____
____
____
____
____
____
____
____
____
____
____
____
____
____

2.6 SWPPP Form

Storm Water Pollution Prevention Plan Narrative

Project Title	
Operator with Control Over Construction Plans and Specifications (Company Name and Address)	
Operator's Representative	Phone No.
Prepared by	Date
I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.	
Signatory Name and Title	Signature

Operator with Day-to-Day Operational Control Over Activities to Ensure Compliance with SWPPP

Company Name and Address	
Operator's Representative	Phone No.

Revisions to SWPPP

Revision No.	Date	Description of Changes	Signature

DISCLAIMER

The North Central Texas Council of Governments or local municipality make no claim regarding the conformity of this form with state or federal regulations. Use of this form does not in any way guarantee that the user will be in compliance with the TPDES Construction General Permit TXR150000. The North Central Texas Council of Governments or local government assume no liability for prosecution arising out of the use of the information and/or SWPPP Narrative form.

For projects requiring permit coverage, the Storm Water Pollution Prevention Plan must be checked thoroughly by the operator for compliance with TPDES Construction General Permit TXR150000.

Copy of NOI(s) or Site Notice(s) and TPDES General Permit TXR150000 attached? ☐

Name of Receiving Water(s)____

Name of Municipal Separate Storm Sewer System (MS4) Receiving Discharge (if applicable) ____

Total Area of Property _________ Acres

Total Area of Site to be Disturbed _________ Acres

Total Area of Off-site Material Storage & Borrow/Fill Sites _______ Acres

Description of Project/Construction Activity

____
____
____
____
____
____

Sequence of Major Construction Activities

Provide a description of the intended sequence of major activities that will disturb soils. Describe the general timing or sequence for implementation (and removal) of controls that will be used to minimize pollution in runoff.

Activity/BMP	Estimated Start	Estimated Completion

Existing Topography and Drainage Features

Describe the existing topography, drainage patterns, and natural drainage features including channels, creeks, watercourses, etc. Provide name (if available) of creeks, streams, etc. and protection measures such as buffers.

____
____
____
____
____

Soil Types

Soil Name	Erosion Factor (K)	Unified Classification	Site Coverage (%)

Existing (Pre-construction) Ground Cover

Describe existing vegetation on the drawing. Such features as tree clusters, grassy areas, and unique or sensitive vegetation should be shown.

____

____

____

Type of Grass/Vegetation/Trees	Approximate Density (%)	Site Coverage (%)

Critical Areas

Describe the location, size, and characteristics of any wetlands, streams, or lakes that are adjacent or in close proximity to the site, and/or will receive discharges from disturbed areas of the project. Also delineate areas with high erosion potential including steep slopes.

____

____

____

____

____

Description of Potential Pollutants

Describe potential pollutants, including construction and waste materials, chemicals, paints, solvents, etc expected to be stored on-site.

____

____

____

____

____

Existing Storm Sewer System

Describe any existing onsite storm sewer systems including location of inlets and outfalls, pipe sizes, etc.

____

____

____

____

____

Permanent (Post-Construction) Storm Water Management Controls

Provide a description of measures that will be installed to control pollutants (sediment, oil, grease, fertilizer, pesticides, etc.) in storm water discharges that will occur after construction is complete and the developed property is placed in service.

____

____

____

____

____

Elements of a Construction SWPPP

For each of the following elements, describe the measures used to address the element. Include the type and location of BMPs used to satisfy the required element and the general timing or sequence for implementation. If an element is not applicable to a project, provide a written justification for why it is not necessary.

1. Limits of Soil Disturbance

Provide a description of the areas including natural drainage features, trees and other vegetation, and appropriate buffers that are to be preserved within the construction area and the measures to be implemented to ensure protection.

____

____

____

____

____

2. Stabilization to Prevent Soil Erosion

Describe the temporary and final stabilization practices for disturbed areas of the site, including a schedule of when the practices will be implemented.

____

____

____

____

____

3. Slope Protection

Describe practices used to protect slopes and divert flows away from exposed soils or disturbed areas.

____
____
____
____
____
____

4. Sediment Barriers and Perimeter Controls

Describe the practices to lessen the off-site transport of sediment and to reduce generation of dust. Sediment basins are required, where feasible, for common drainage locations that serve an area with ten or more acres disturbed at one time.

____
____
____
____
____
____

5. Velocity Dissipation and Chanel Stabilization

Provide a description of velocity dissipation devices used at discharge locations and channel stabilization measures to provide non-erosive flows.

____
____
____
____
____
____

6. Construction Access Controls

Provide a description of measures to minimize the off-site tracking of sediment by vehicles.

____
____
____
____
____
____

7. Storm Drain Inlet Protection

Provide a description of inlet protection measures to prevent sediment from entering the storm drain system.

____
____
____
____
____
____

8. Dewatering Controls

Provide a description of controls to prevent the off-site transport of suspended sediments and other pollutants in discharges from dewatering operations.

____
____
____
____
____
____

9. Material and Waste Controls

Provide a description of controls to reduce pollutants and spill prevention and response procedures associated with construction and waste materials. Also provide a description of controls and measures that will be implemented to minimize pollutants in any discharges associated with industrial activity other than construction (i.e., dedicated asphalt or concrete plants) covered by the TPDES Construction General Permit.

____
____
____
____
____
____

10. Construction Phasing and Project Management

Provide a description of considerations given to project phasing in order to reduce the amount of soil exposed at one time.

____
____
____
____
____
____

Hydrology:

1.0 Hydrological Analysis
2.0 Downstream Assessment
3.0 Streambank Protection
4.0 Water Balance
5.0 Rainfall Tables
6.0 Hydrologic Soils Data

Table of Contents

List of Tables

List of Figures

1.0 Hydrological Analysis

1.1 Estimating Runoff

1.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater controls. In the hydrologic analysis of a development/redevelopment site, there are a number of variable factors that affect the nature of stormwater 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 1.1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 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 1.1 Applications of the Recommended Hydrologic Methods

Method	Technical 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)	Section 1.2 of Water Quality						✓
Streambank Protection Volume (SP_v)	Section 3.0 of Hydrology		✓		✓		
Flood Mitigation Discharge (Q_f)	Section 1.3 of Criteria Manual		✓		✓	✓	
Storage Facilities	Section 2.0 of Hydraulics		✓	✓	✓		✓
Outlet Structures	Section 2.2 of Hydraulics		✓		✓		
Gutter Flow and Inlets	Section 1.2 of Hydraulics	✓					
Storm Drain Pipes	Section 1.1 of Hydraulics	✓	✓		✓		
Culverts	Section 3.3 of Hydraulics	✓	✓		✓	✓	
Bridges	Section 3.4 of Hydraulics		✓		✓		
Small Ditches	Section 3.2 of Hydraulics	✓	✓		✓		
Open Channels	Section 3.2 of Hydraulics		✓		✓	✓	
Energy Dissipation	Section 4.0 of Hydraulics		✓		✓		

Table 1.2 Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 – 100 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Modified Rational[2]	0 – 200 acres	Method can be used for estimating runoff volumes for storage design.
Unit Hydrograph (SCS)[3]	Any Size	Method can be used for estimating peak flows and hydrographs for all design applications.
Unit Hydrograph (Snyder's)[4]	1 acre and larger	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 can be used for calculating the Water Quality Protection Volume (WQ_v).

[1] Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
[2] Where the Modified Rational Method is used for conceptualizing, the engineer is cautioned that the method could underestimate the storage volume.
[3] This refers to SCS routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.
[4] This refers to the Snyder's 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.

1.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 1.3 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 1.3 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 1.3 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

1.1.3 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 16 counties which participate in the NCTCOG area (see Figure 1.1) are provided in *Section 5.0* 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:

  $$i = b/(t + d)^e \qquad (1.1)$$

  where:
  i = rainfall intensity (inches per hour)
  t = rainfall duration (minutes)
  b, d and e = parameters found at the top of each of the tables in Section 5.0.
- The tabular values in *Section 5.0* Rainfall Tables were determined from the new IDF curves.

Figure 1.2 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.

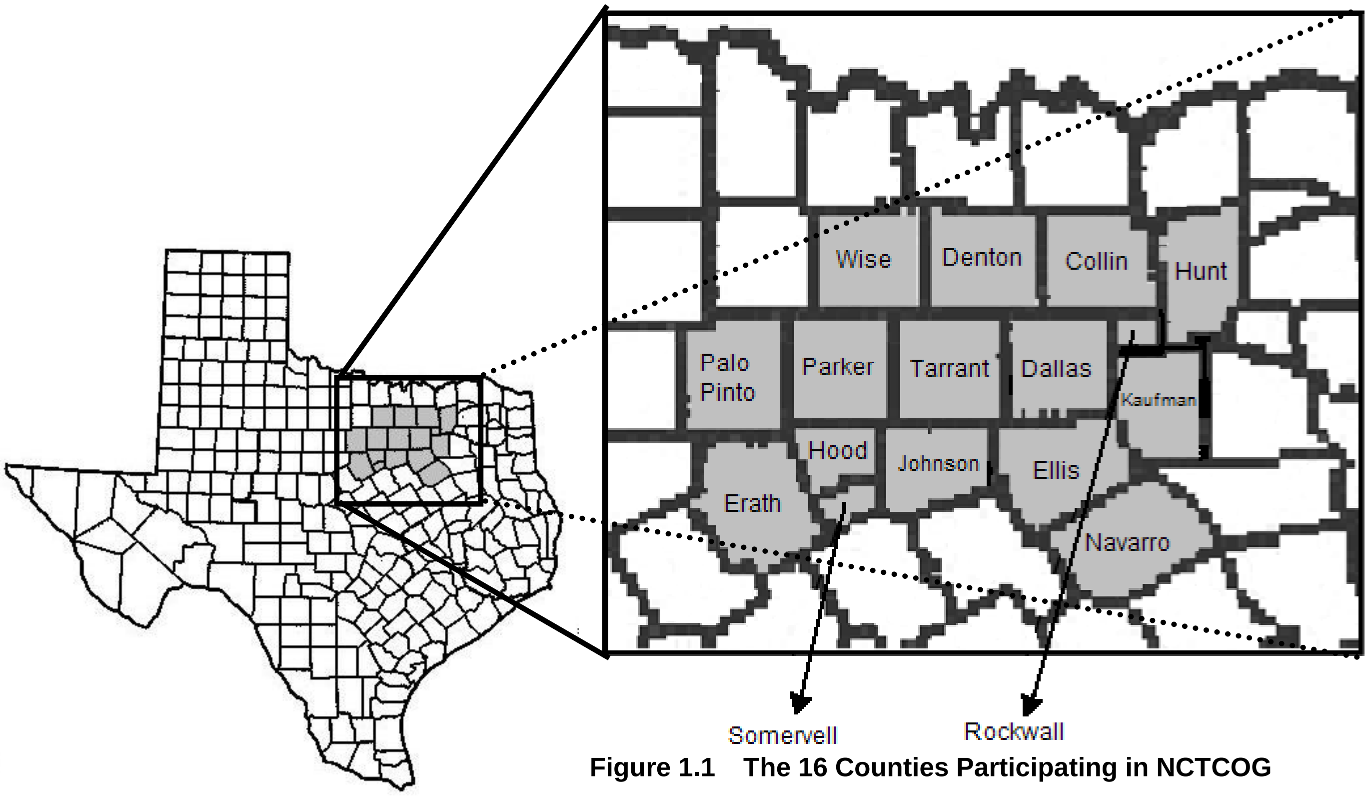


Figure 1.1 The 16 Counties Participating in NCTCOG

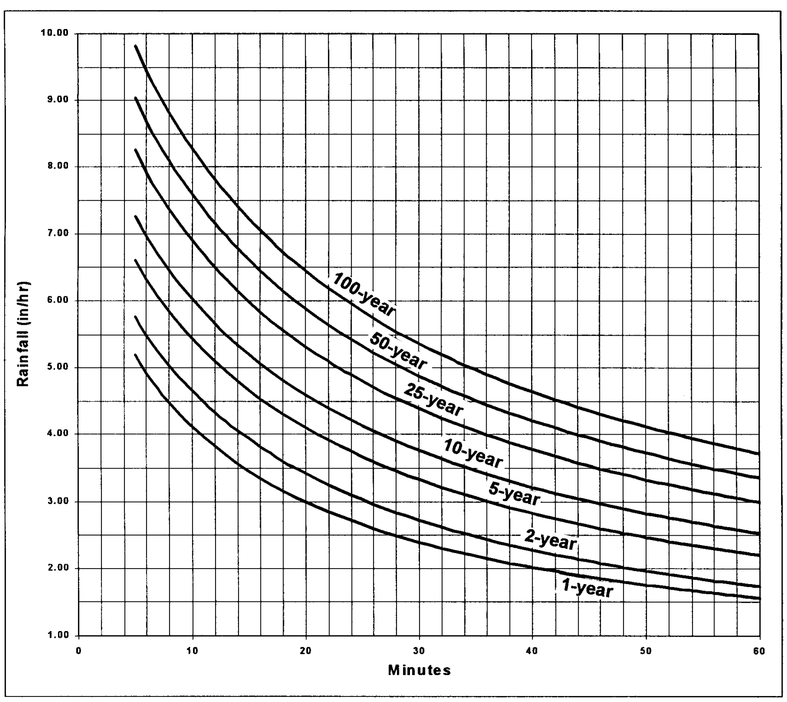


Figure 1.2 Example IDF Curve (Dallas County, Texas)

1.2 Rational Method

1.2.1 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.

1.2.2 Application

The Rational Method can be used to estimate stormwater 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 *Section 1.5*. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in *Section 1.5* 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.

1.2.3 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:

$$Q = CIA \tag{1.2}$$

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 1.6 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:

$$Q = C_f CIA \tag{1.3}$$

The C_f values that can be used are listed in Table 1.4. The product of C_f times C shall not exceed 1.0.

Table 1.4 Frequency Factors for Rational Formula

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

1.2.4 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 1.3 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 1.3, the formula often used, which seems to match the nomograph very closely, is as follows:

$$T_c = 1.8(1.1 - C)(D)^{0.5}/(S)^{(1/3)} \tag{1.4}$$

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 1.4, and (2) Figure 1.3: D = 250 ft, C = 0.7, S = 0.50% slope.

1. Figure 1.3 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 1.4. More guidance on travel time estimation is given in *Section 1.3.6*.

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.

Table 1.5 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.

Table 1.5 Times of Concentration

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.

1.2.5 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 *Section 5.0*.

1.2.6 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 1.6 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 1.6 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.

1.2.7 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.

<u>Site Data</u>

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

<u>Overland Flow</u>

A runoff coefficient (C) for the overland flow area is determined from Table 1.6 to be 0.10.

Table 1.6 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

Time of Concentration

From Figure 1.3 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 1.4 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 5.15 in *Section 5.0*, 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 1.7.

Table 1.7 Weighted Runoff Coefficient

1	2	3	4
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 4 equals column 2 multiplied by column 3.			

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

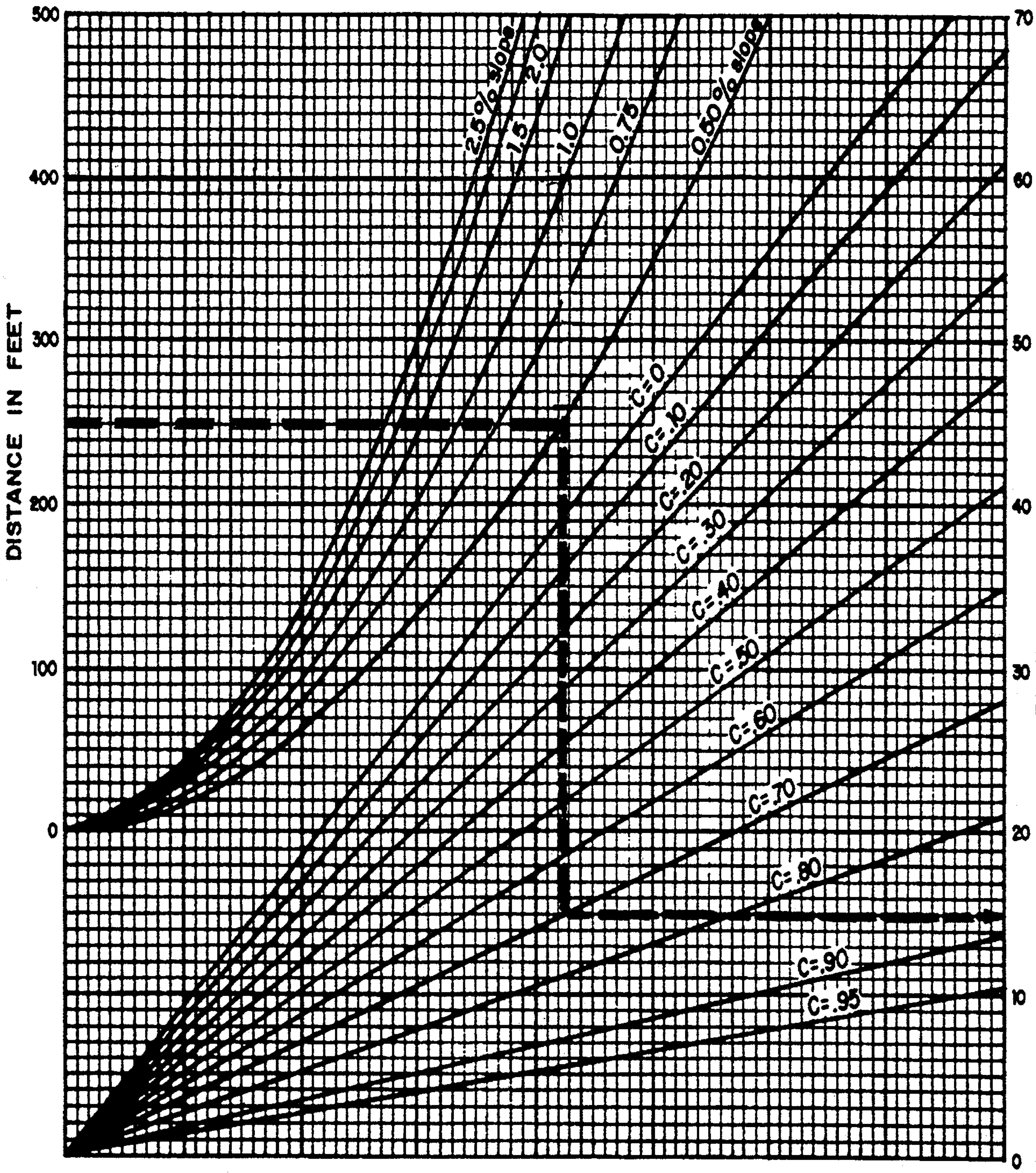
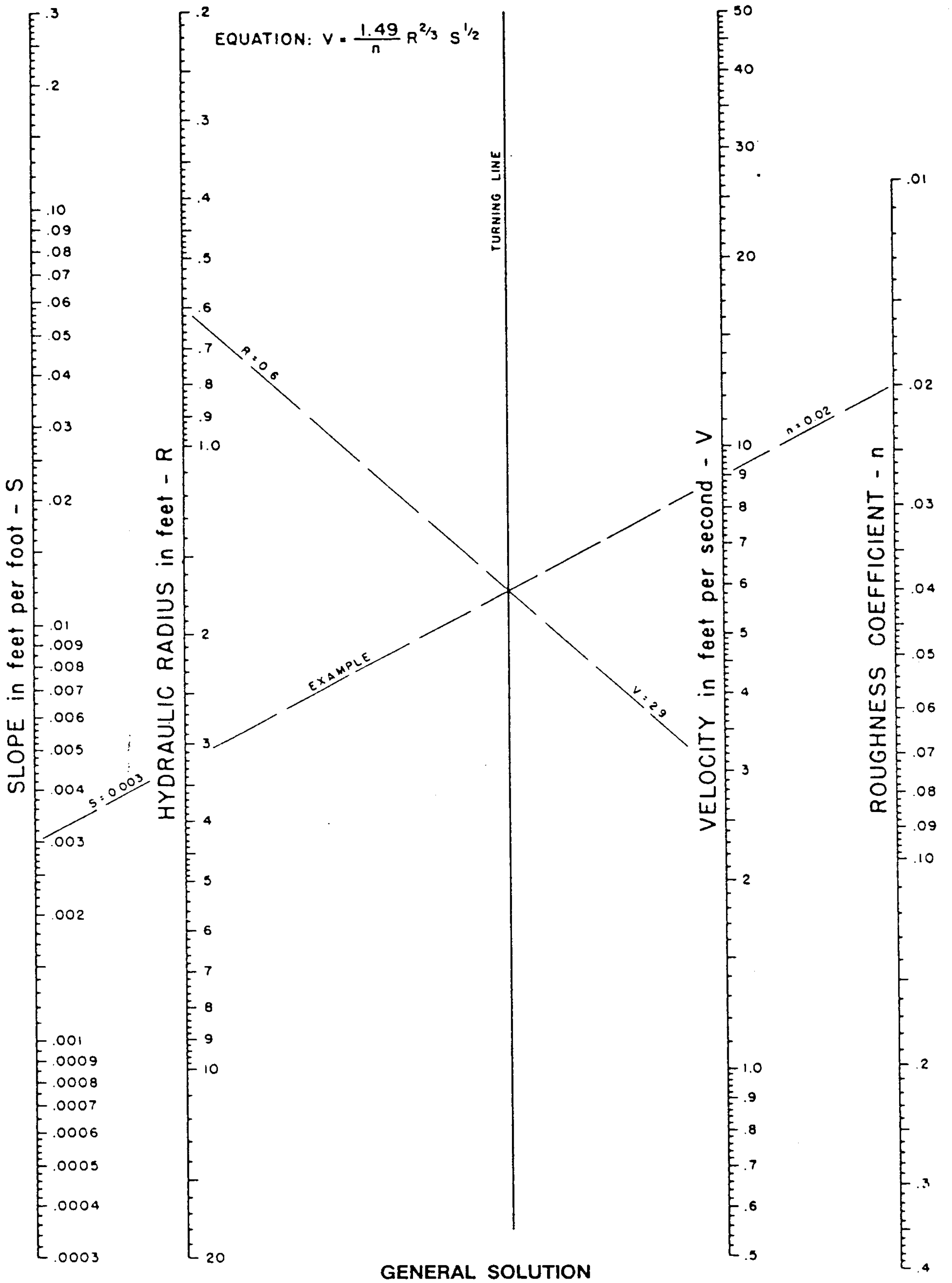


Figure 1.3 Rational Formula - Overland Time of Flow Nomograph
(Source: Airport Drainage, Federal Aviation Administration, 1965)



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

Figure 1.4 Manning's Equation Nomograph
(Source: USDOT, FHWA, HDS-3 (1961))

1.3 SCS Hydrological Method

1.3.1 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 1.5 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.

1.3.2 Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified method of *Section 1.3.7* 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 dissipaters.

1.3.3 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 1.5).

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

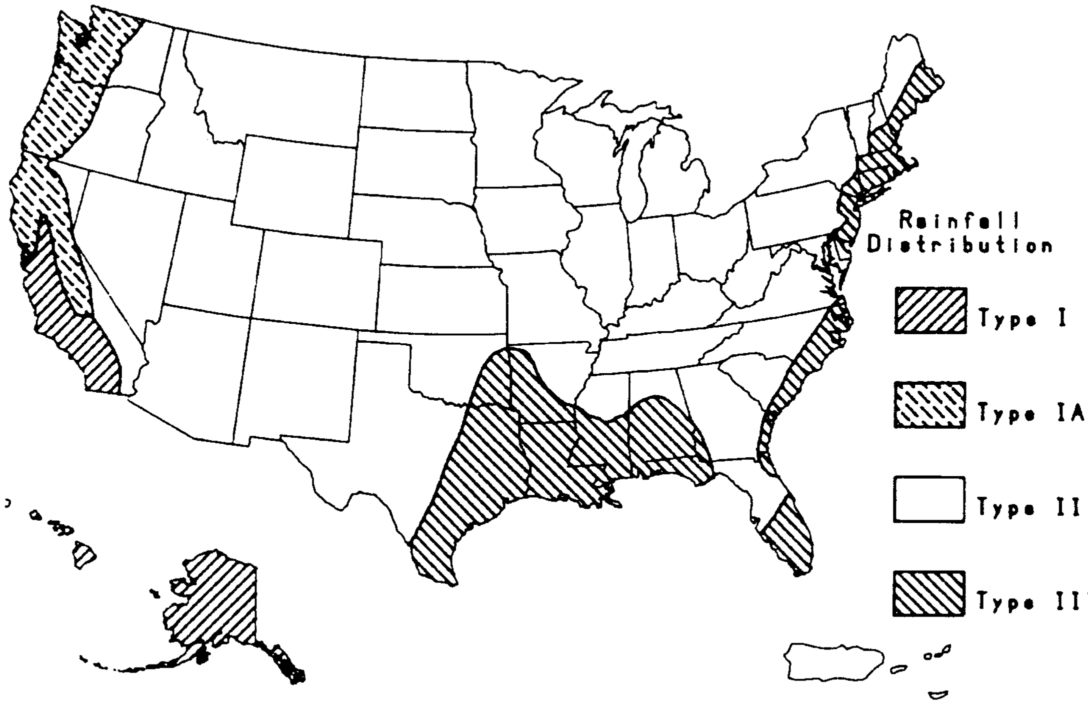


Figure 1.5 Approximate Geographic Boundaries for SCS Rainfall Distributions

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:

$$\mathbf{Q = (P - I_a)^2 / [(P - I_a) + S]} \qquad \textbf{(1.5)}$$

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:

$$\mathbf{I_a = 0.2S} \qquad \textbf{(1.6)}$$

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 1.11 provides values of I_a for a wide range of curve numbers (CN).

Substituting 0.2S for I_a in Equation 1.6, the equation becomes:

$$\mathbf{Q = (P - 0.2S)^2 / (P + 0.8S)} \qquad \textbf{(1.7)}$$

Figure 1.6 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.

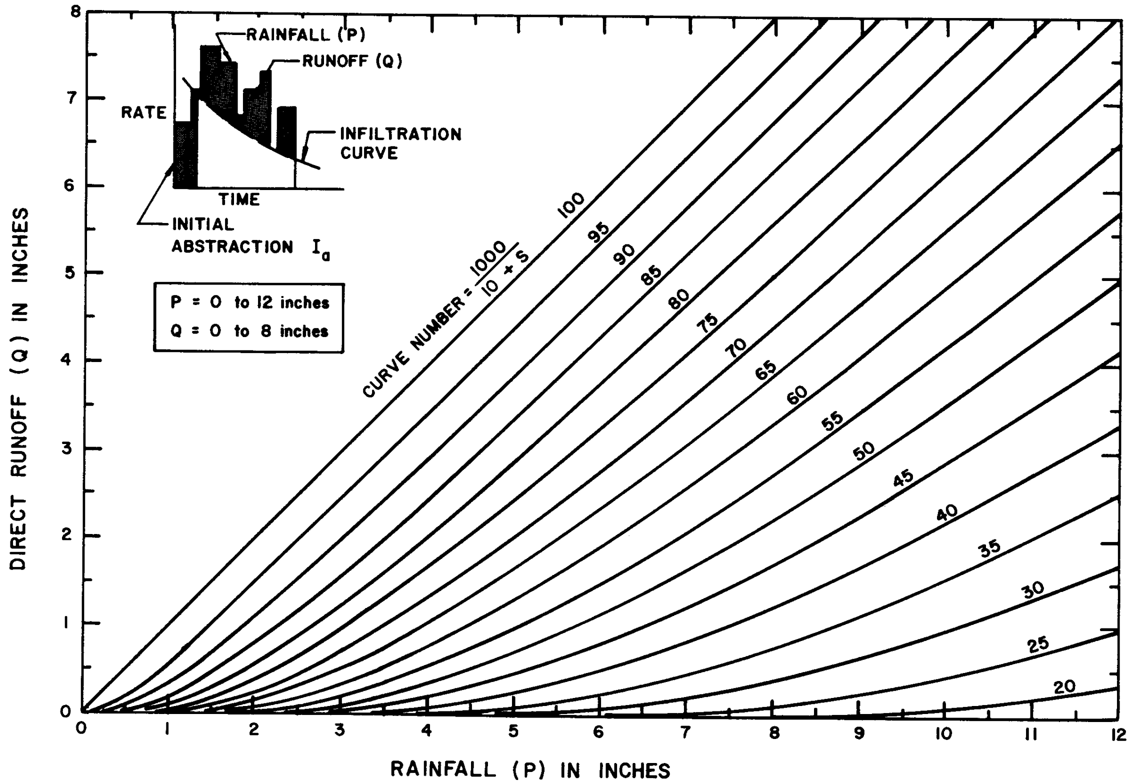


Figure 1.6 SCS Solution of the Runoff Equation
(Source: SCS, TR-55, Second Edition, June 1986)

Equation 1.7 can be rearranged so the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \qquad (1.8)$$

1.3.4 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. *Section 6.0 - Hydrologic Soils Data* 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 1.9 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 in Table 1.8.

Table 1.8 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.

1.3.5 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 1.9 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 1.9 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 1.9 are not applicable, use Figure 1.7 to compute a composite CN. For example, Table 1.9 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 1.7 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

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 1.8 if total impervious area is less than 30% or (2) use Figure 1.7 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 1.8 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 1.8 is 66. If all of the impervious area is connected, the resulting CN (from Figure 1.7) would be 68.

1.3.6 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).

<u>Travel Time</u>

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:

$$\mathbf{T_t = L/3600V} \qquad \textbf{(1.9)}$$

where:

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

Table 1.9 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
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.

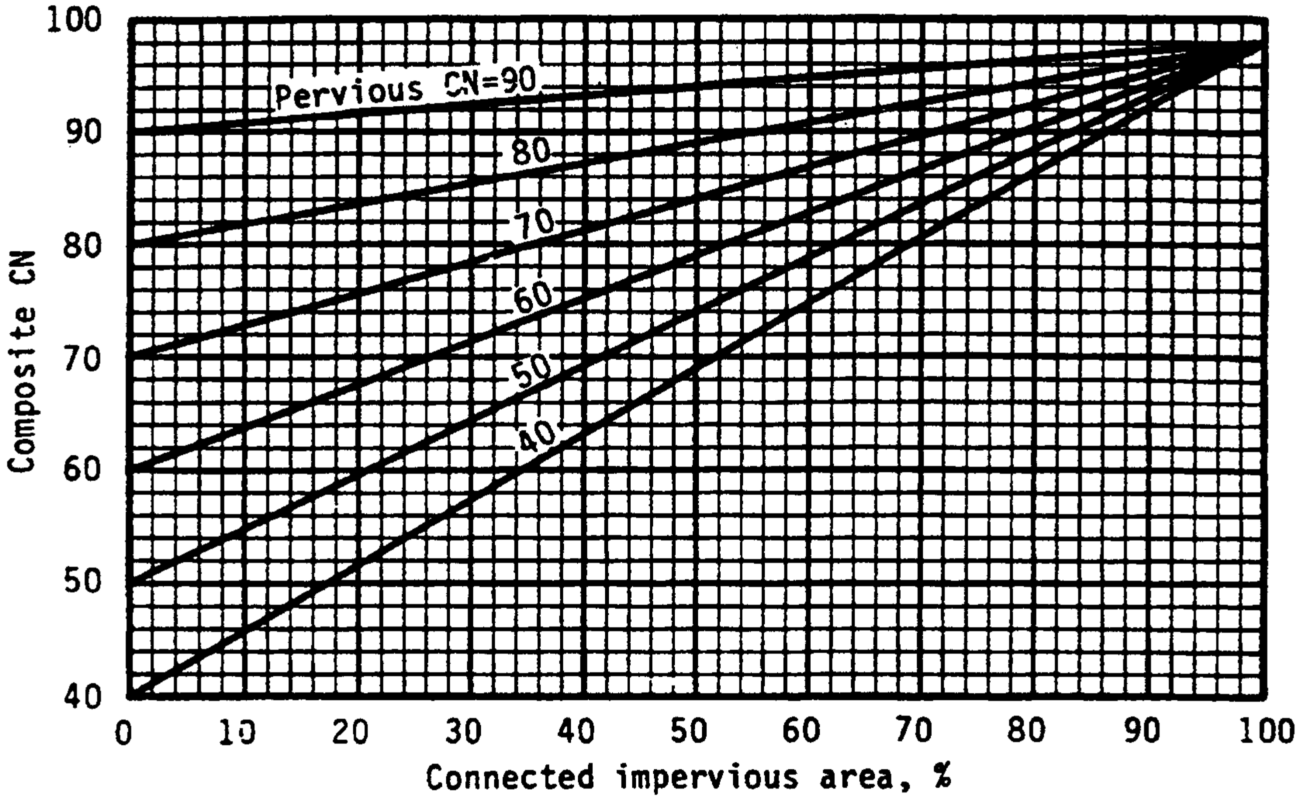


Figure 1.7 Composite CN with Connected Impervious Areas
(Source: SCS, TR-55, Second Edition, June 1986)

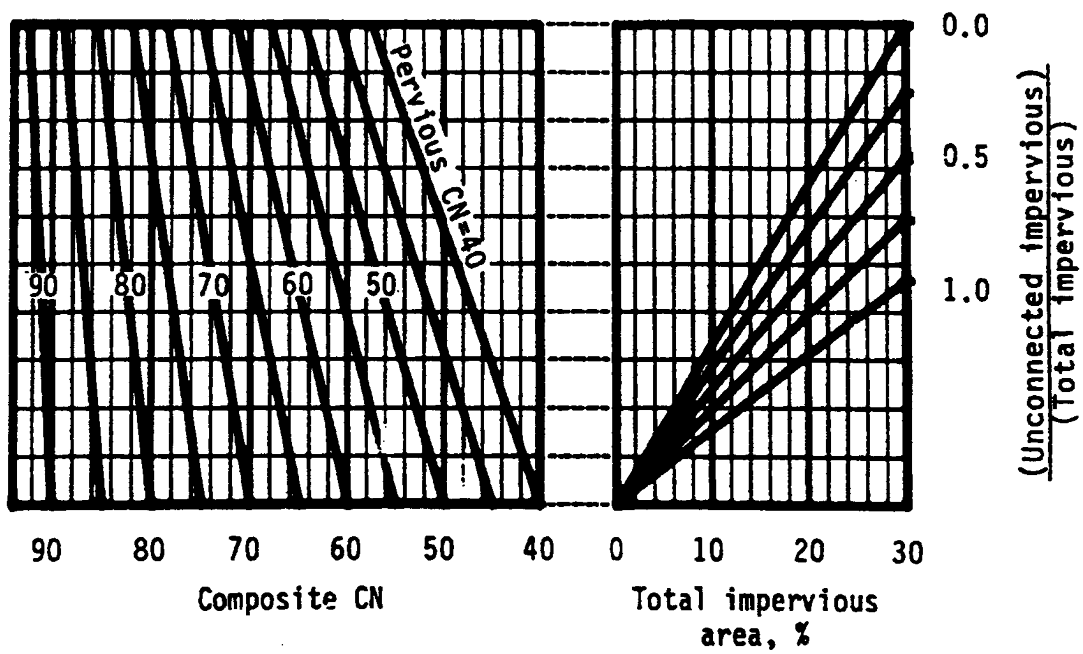


Figure 1.8 Composite CN with Unconnected Impervious Areas (Total Impervious Area Less Than 30%)
(Source: SCS, TR-55, Second Edition, June 1986)

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} \qquad (1.10)$$

where:

T_t = travel time (hr)
n = Manning roughness coefficient (see Table 1.10)
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall (in)
S = land slope (ft/ft)

Table 1.10 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 1.9, 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 1.9, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $V = 16.13(S)^{0.5}$ **(1.11)**

Paved $V = 20.33(S)^{0.5}$ **(1.12)**

where:

V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 1.9 or Equations 1.11 or 1.12, use Equation 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

$$\mathbf{V = (1.49/n)\,(R)^{2/3}\,(S)^{1/2}} \qquad \mathbf{(1.13)}$$

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 1.13, T_t for the channel segment can be estimated using Equation 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.

1.3.7 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:

$$\mathbf{Q_p = q_u A Q F_p} \qquad \mathbf{(1.14)}$$

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

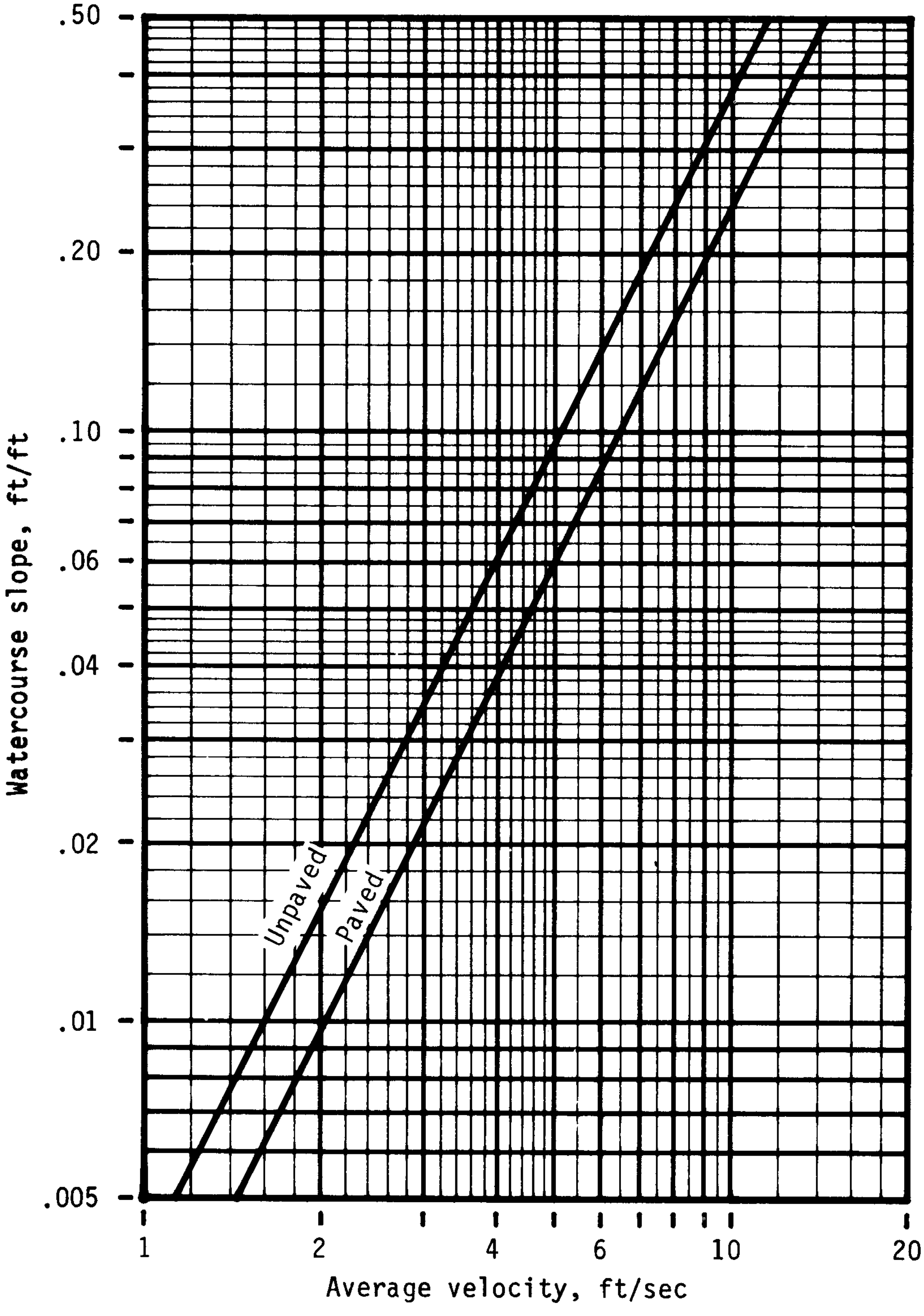


Figure 1.9 Average Velocities - Shallow Concentrated Flow
(Source: SCS, TR-55, Second Edition, June 1986)

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth (P) is determined from the rainfall tables in *Section 5.0* for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 1.9 and direct runoff, Q, is calculated using Equation 1.7.
3. The CN value is used to determine the initial abstraction, I_a, from Table 1.11, 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 *Section 1.3.6* and is used with the ratio I_a/P to obtain the unit peak discharge (q_u) from Figure 1.10 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 1.10 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 1.3.9*.
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 1.14.

1.3.8 Example Problem 1

Compute the flood mitigation storm 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 flood mitigation storm, 24-hour rainfall is 9.12 inches (.38 in/hr x 24 hours – From *Section 5.0*, Table 5.15).
- 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 (flood mitigation storm) = 7.1 inches

Table 1.11 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

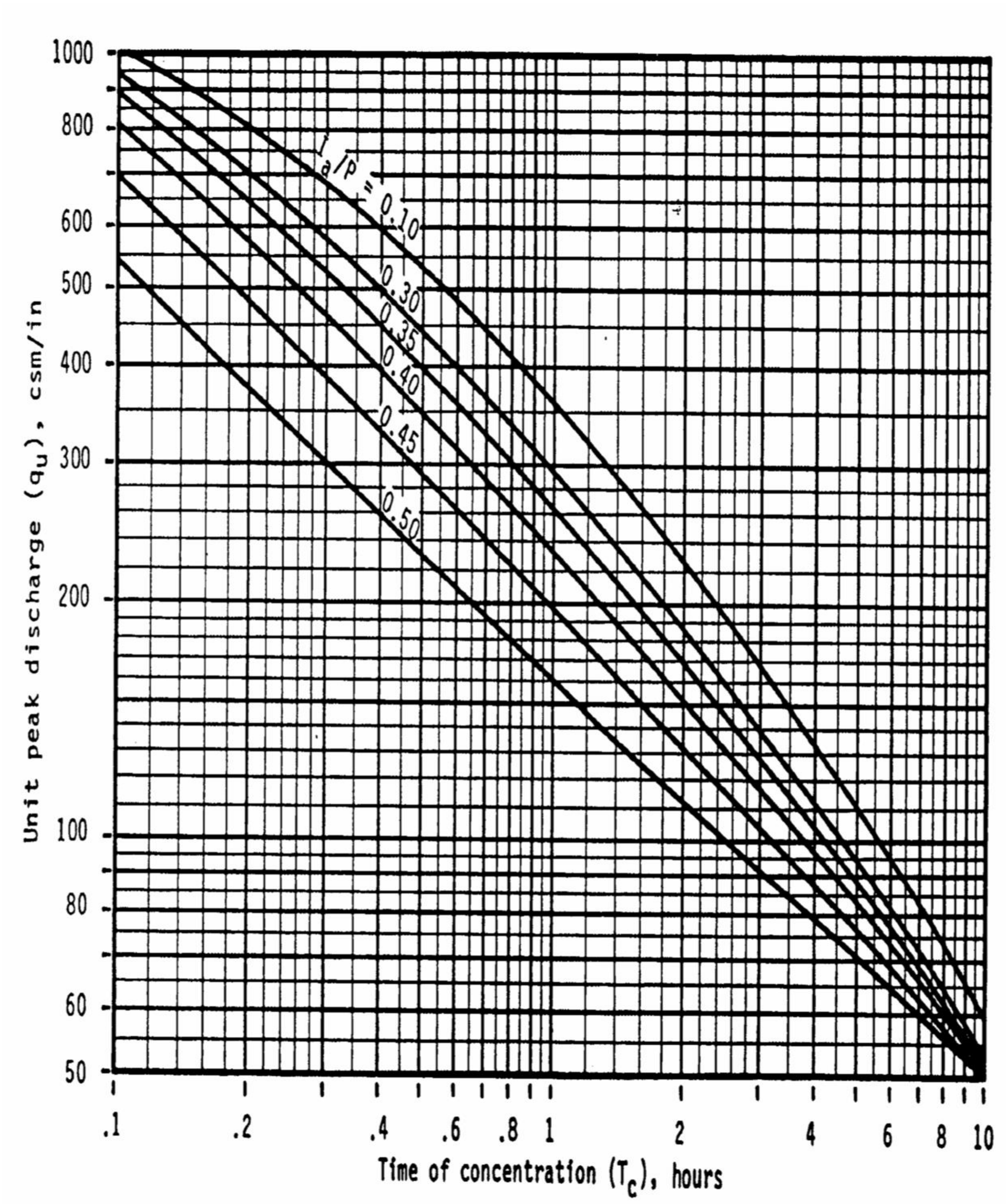


Figure 1.10 SCS Type II Unit Peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

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 1.10 with P_2 = 3.36 inches
   (0.14 x 24 – *Section 5.0*, Table 5.15)

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

   Segment 2 - Travel time from Figure 1.9 or Equation 1.10
   V = 2.1 ft/sec (from Equation 1.11)
   T_t = 750 / 60 (2.1) = 5.95 minutes

   Segment 3 - Using Equation 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 1.9), I_a = .410 (Table 1.11)

   I_a/P = (.410 / 9.12) = .05
   (Note: Use I_a/P = .10 to facilitate use of Figure 1.10.)

4. Unit discharge q_u (flood mitigation storm) from Figure 1.10 = 650 csm/in

5. Calculate peak discharge with F_p = 1 using Equation 1.14

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

1.3.9 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 *Section 1.3.3* 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 *Sections 1.3.4, 1.3.5, and 1.3.6*.
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 1.8).
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 1.11 and Table 1.12 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:

$$q_u = (PF*A) / (T_p) \qquad (1.15)$$

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:

$$\frac{q}{q_u} = \left(\frac{t}{T_p}\, e^{[1-(t/T_p)]}\right)^X \qquad (1.16)$$

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

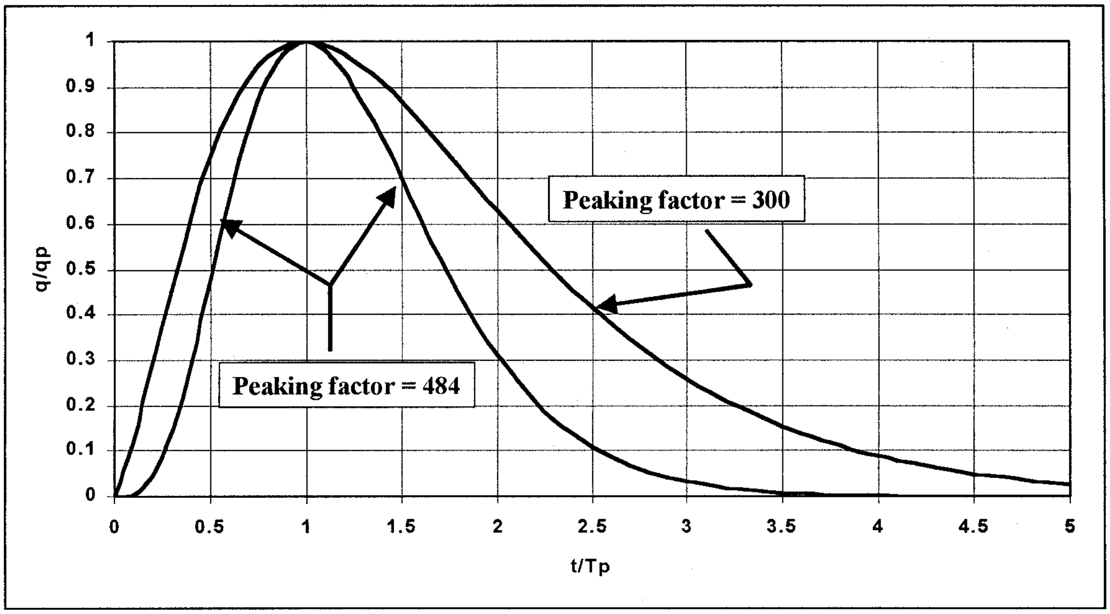


Figure 1.11 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 1.12 Dimensionless Unit Hydrograph With Peaking Factor of 484

t/T_t	484	
	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

1.3.10 Example Problem 2

Compute the unit hydrograph for the 50-acre watershed in Example Problem 1 (*Section 1.3.8*).

Computations

1. 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)

2. Calculate q_{pu}

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

3. Calculate unit hydrograph.

   Based on spreadsheet calculations using Equations 1.15 and 1.16, Table 1.13 has been derived.

Table 1.13 Example of Dimensionless Unit Hydrograph With Peaking Factor of 484

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

1.3.11 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).

1.4 Snyder's Unit Hydrograph Method

1.4.1 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.

1.4.2 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:

$$\mathbf{t_p = C_t\,(L\,L_{ca})^{0.3}} \qquad \mathbf{(1.17)}$$

$$\mathbf{t_r = t_p \div 5.5} \qquad \mathbf{(1.18)}$$

$$\mathbf{q_p = C_p 640 \div t_p} \qquad \mathbf{(1.19)}$$

$$\mathbf{t_{pR} = t_p + 0.25(t_R - t_r)} \qquad \mathbf{(1.20)}$$

$$\mathbf{q_{pR} = C_p 640 \div t_{pR}} \qquad \mathbf{(1.21)}$$

$$\mathbf{q_{pR} = q_p\, t_p \div t_{pR}} \qquad \mathbf{(1.22)}$$

$$\mathbf{Q_p = q_p\, A} \qquad \mathbf{(1.23)}$$

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 1.14.

Table 1.14 Typical Values of C_p

Typical Drainage Area Characteristics	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

1.4.3 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:

$$t_p = 10\text{^}[0.3833*\log_{10}(L*L_{ca}/(S_{st})^{0.5})+(\log_{10}(Ip))-BW*(\%Urb/100)] \quad (1.24)$$

$$S_{st} = (el_{85\%} - el_{15\%})/(0.7*L) \quad (1.25)$$

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.

1.4.4 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.

Percent Urbanization

Urbanization is defined as the percentage of the basin which has been developed and improved with channelization and/or a stormwater 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 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 1.15. 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 1.6* in the application of USGS regression equations to determine peak flows for urban basins.

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 (Table 1.16) 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 1.15 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

Table 1.16 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, as shown in Table 1.17, 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 1.17 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 1.3.11*, for an explanation of routing methods and references for further information.

1.5 Modified Rational Method

1.5.1 Introduction

For drainage areas of *less than 200 acres*, a modification of the Rational Method can be used for the estimation or design 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 1.12

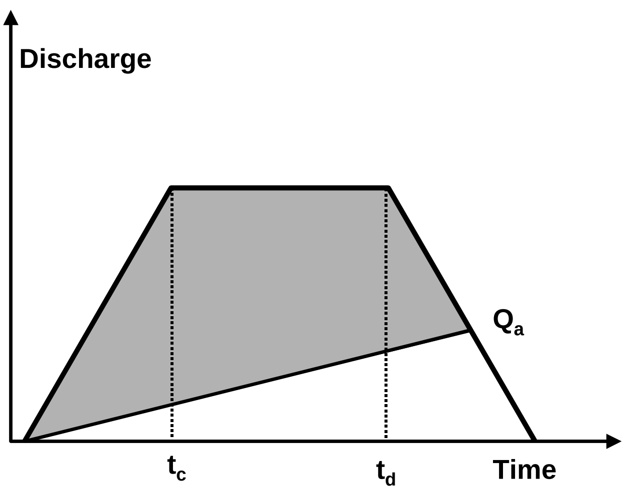


Figure 1.12 Modified Rational Definitions

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.

1.5.2 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:

$$\mathbf{Q_a = C_a\ i\ A} \qquad \mathbf{(1.26)}$$

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:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \qquad \mathbf{(1.27)}$$

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 1.18

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

$$\mathbf{V_{preliminary} = 60\ [CAa - (2CabAQ_a)^{1/2} + (Q_a/2)\ (b - t_c)]} \qquad \mathbf{(1.28a)}$$

$$\mathbf{V_{max} = V_{preliminary} * P_{180}/P_{td}} \qquad \mathbf{(1.28b)}$$

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 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 1.18 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

1.5.3 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 flood mitigation 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 *Section 5.0*. 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 1.26) (cfs)	3.69	5.27
a (from Table 1.18)	128.85	210.73
b (from Table 1.18)	16.624	21.785
V_{pre} (Equation 1.28a) (ft^3)	16,570	26,042
P_{180} (in)	2.28	3.60
T_d (Equation 1.27) (min)	51.52	61.69
P_{td} (in)	1.65	2.66
V_{max} (Equation 1.28b) (ft^3)	22,897	35,245

1.6 USGS and TxDOT Regression Methods

1.6.1 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.

1.6.2 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

1.6.3 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 1.19 gives the USGS regression equations for urban streams in the Dallas-Fort Worth area.

1.6.4 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 1.19 USGS Peak Flow Regression Equations for Dallas-Fort Worth Urban Areas	
Frequency	**Equations**
2-year	$Q_2 = 42.83(A)^{0.704}(UI)^{0.836}$
5-year	$Q_5 = 82.92(A)^{0.724}(UI)^{0.751}$
10-year	$Q_{10} = 120.7(A)^{0.735}(UI)^{0.697}$
25-year	$Q_{25} = 184.8(A)^{0.745}(UI)^{0.632}$
50-year	$Q_{50} = 246.4(A)^{0.752}(UI)^{0.587}$
100-year	$Q_{100} = 362.1(A)^{0.752}(UI)^{0.510}$
For these equations: A = drainage area in mi^2, UI = urbanization index Source: USGS, 1982	

1.6.5 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.

1.6.6 Rural (or Undeveloped) Basin Application

Equation 1.29 applies to rural, uncontrolled watersheds. Figure 1.13 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 1.20 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.

$$\mathbf{Q_T = aA^bSH^cSL^d} \qquad \mathbf{(1.29)}$$

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 1.20.

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 1.30.

$$\mathbf{Q_w = (2 - \log(A/z))Q_1 + (\log(A/z)-1)Q_2} \qquad \mathbf{(1.30)}$$

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)

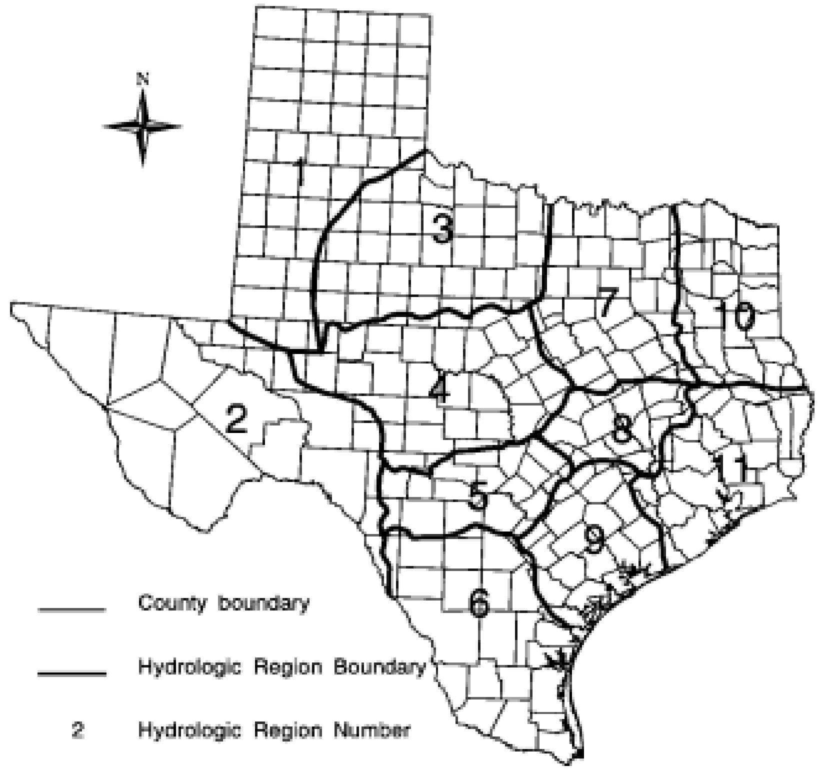


Figure 1.13 Hydrologic Regions for Statewide Rural Regression Equations
Source: TXDOT, 2002

Table 1.20 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]		

Table 1.20 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)
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} =	$1842\ A^{.558}$	
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 1.29 to calculate a weighted discharge for streams with contributing drainage area falling within the arrange of 10 to 100 square miles.

1.6.7 Example Problem

For the 100-year storm, 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

Peak Discharge Calculations

The 100-year storm Rural Peak Discharge determination for Region 7 will necessitate the use of Equation 1.30 because the drainage area is in the range of 10-100 square miles. The first step is to determine the discharge based on regression coefficients for areas greater than 32 square miles and less than 32 square miles. Table 1.20 provides the regression equations for Region 7 as follows;

For contributing drainage area less than 32 square miles,

$$\begin{aligned} Q_1 &= 1842\ A^{.558} \\ &= 1842(38.8)^{.558} \\ &= 14{,}186 \text{ cfs} \end{aligned}$$

For contributing drainage area greater than 32 square miles,

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

Equation 1.30 is then used to determine the 100-year storm Rural Peak Discharge.

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

1.7 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 Stormwater Management. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

Ferguson, B. and Debo, T., 1990. On-Site Stormwater Management.

Hershfield, D. M., 1961. Rainfall Frequency Atlas of the United States, Technical Paper 40.

Maryland Department of the Environment, 2000. Maryland Stormwater 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

2.0 Downstream Assessment

2.1 Introduction

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.

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 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
    - "Streambank Protection" storm
    - "Conveyance" storm
    - "Flood Mitigation" storm
- 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.

2.2 Downstream Hydrologic Assessment

Common practice requires 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 stormwater peak flows and volumes that a community may require as part of a developer's stormwater management site plan.

2.3 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. 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.

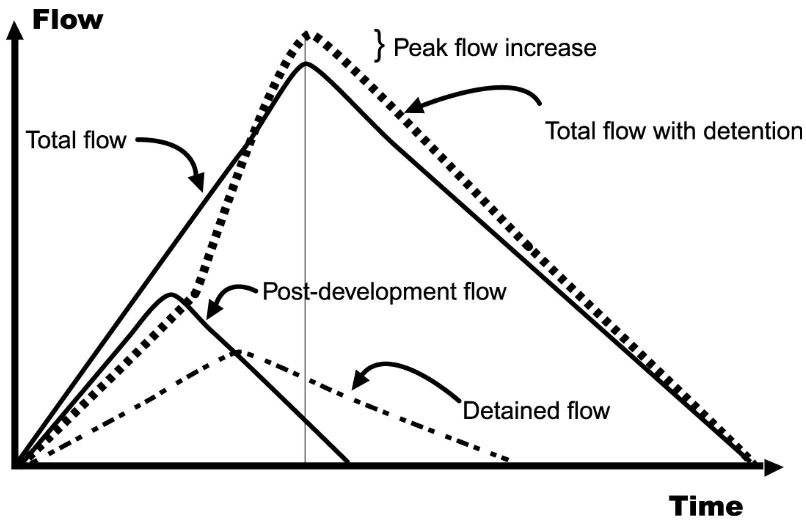


Figure 2.1 Detention Timing Example

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 2.2 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.

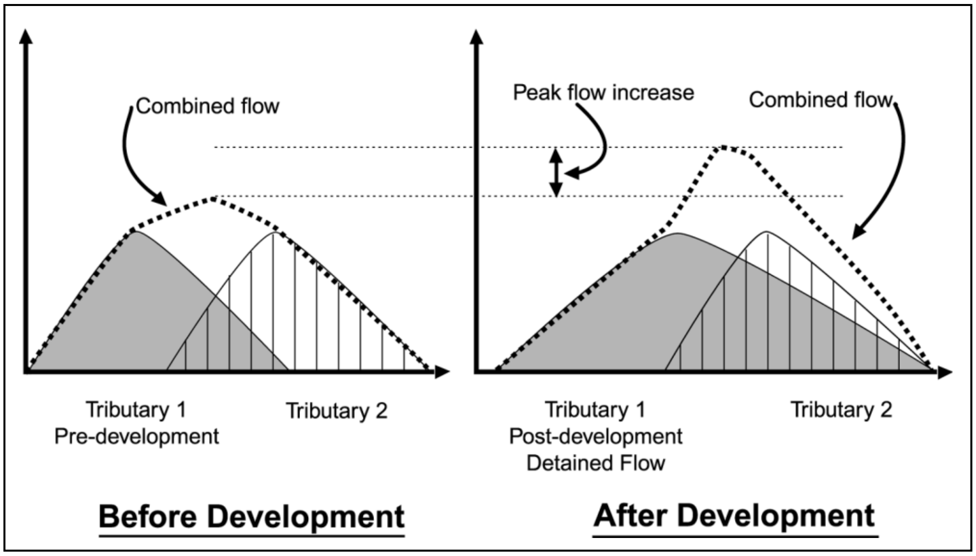


Figure 2.2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

2.4 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on stormwater 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 stormwater 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

- "Conveyance" storm
- "Flood Mitigation" storm

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 3.2 of the Hydraulics Technical 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 stormwater 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.

2.5 Example Problem

Figure 2.3 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) stormwater 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 <u>increase</u> the peak flow in the stream.

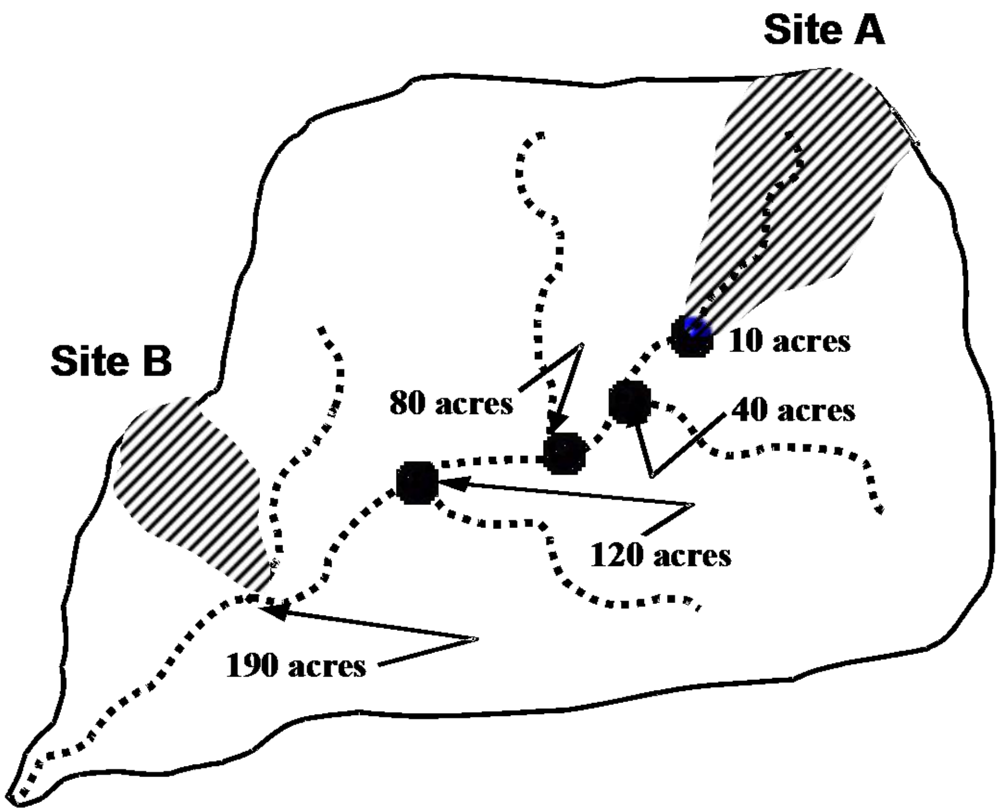


Figure 2.3 Example of the Ten-Percent Rule

3.0 Streambank Protection Volume Estimation

3.1 Streambank Protection Volume Calculation

The Simplified SCS Peak Runoff Rate Estimation approach (see *Section 1.3.7*) 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 1.5* for preliminary detention calculations without formal routing or the SCS Hydrologic Method in *Section 1.3.*

For SP_v estimation, using Figure 1.10, 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 3.1.

Using the following equation from TR-55 for a Type II rainfall distribution, V_S/V_r can be calculated.

Note: Figure 3.2 can also be used to estimate V_S/V_r.

$$V_S/V_r = 0.682 - 1.43\,(q_O/q_i) + 1.64\,(q_O/q_i)^2 - 0.804\,(q_O/q_i)^3 \tag{3.1}$$

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:

$$V_S = \frac{(V_S/V_r)(Q_d)(A)}{12} \tag{3.2}$$

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.

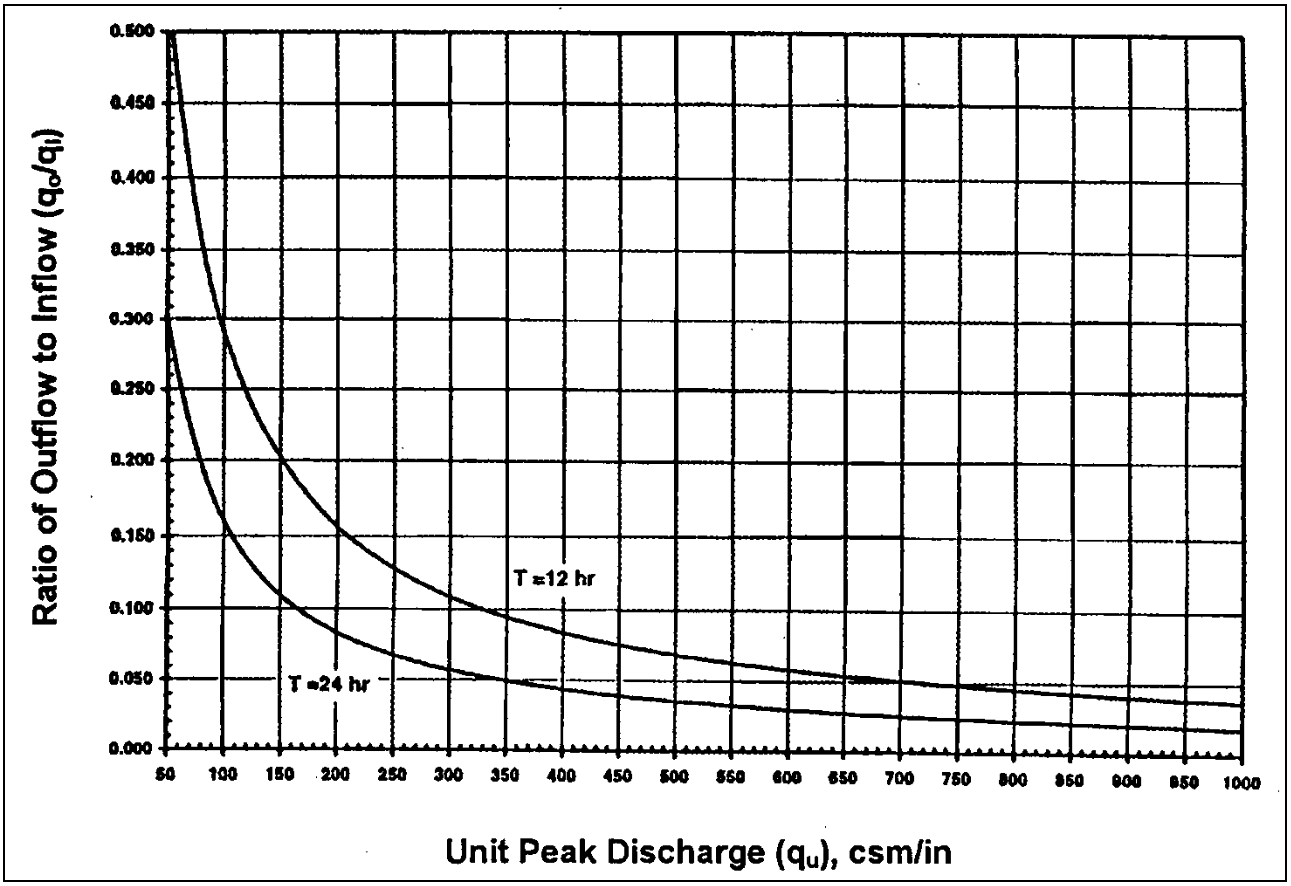


Figure 3.1 Detention Time vs. Discharge Ratios
(Source: MDE, 1998)

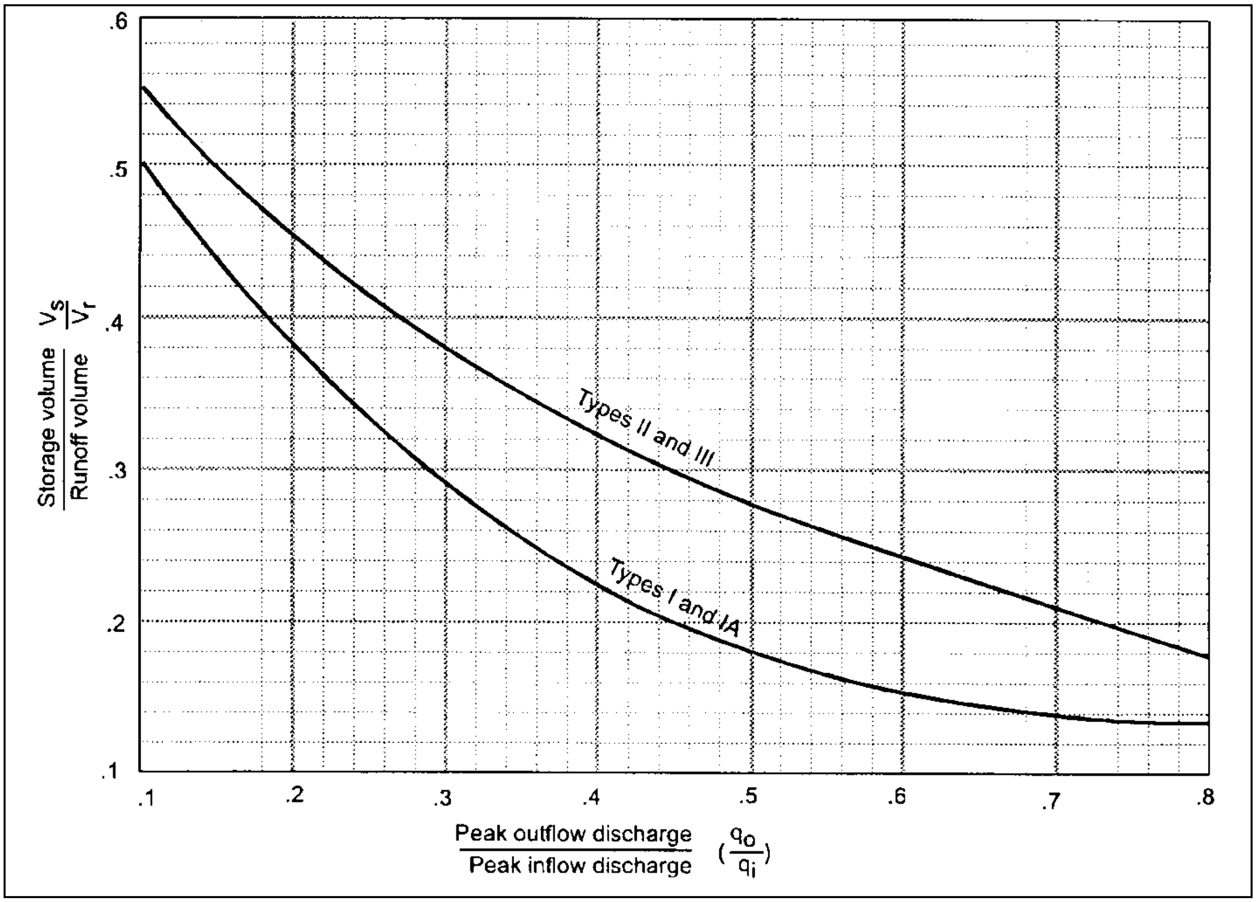


Figure 3.2 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
(Source: TR-55, 1986)

3.2 Example Problem

Compute the Streambank Protection Volume (SP_v) for the 50-acre watershed in *Section 1.3.8 Example Problem One.*

Computations

1. Calculate rainfall excess:
   - The 1-year, 24 hour rainfall is 2.64 inches (0.11 in/hr x 24 hours – From Table 5.16).
   - Composite area-weighted Curve Number is 83.
   - From Equation 2.1.7, Q_d (1-year developed) = 1.2 inches

2. Calculate time of concentration

   t_c = 20.86 minutes (.35 hours)

3. Calculate I_a/P for CN = 83; I_a = .410 (Table 1.11)

   I_a/P = (.410 / 2.64) = .155 (Note: Use straight-line interpolation to facilitate use of Figure 1.10)

4. Find unit discharge q_u:

   From Figure 1.10 for I_a/P = .155 and t_c = .35 hr

q_u = 600 csm/in

5. Find discharge ratio q_O/q_I:

   From Figure 3.1 for q_u = 600 csm/in and T = 24 hr
   $q_O/q_I = 0.03$

6. 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

4.0 Water Balance

4.1 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.

4.2 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):

$$\Delta V = \Sigma I - \Sigma O \qquad (4.1)$$

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 4.1 can be changed to reflect these factors.

$$\Delta V = P + R_o + B_f - I - E - E_t - O_f \qquad (4.2)$$

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 4.4)

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 4.1 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 1.1 of the Water Quality Technical Manual 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 4.3 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$R_o = 0.9(P/12)R_vA \tag{4.3}$$

where:

P = precipitation (in)

R_o = runoff volume (acre-ft)

R_v = volumetric runoff coefficient [see *Equation 1.1 of the Water Quality Technical Manual*]

A = Area in acres

Table 4.1 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 stormwater 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:

$$I = Ak_hG_h \tag{4.4}$$

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 4.2 can be used.

Table 4.2 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 Stormwater 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 4.3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information for Grapevine, Texas. Figure 4.1 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 stormwater ponds and wetlands being designed in Texas. Total annual values can be estimated from this map and distributed according to Table 4.3.

Table 4.3 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.

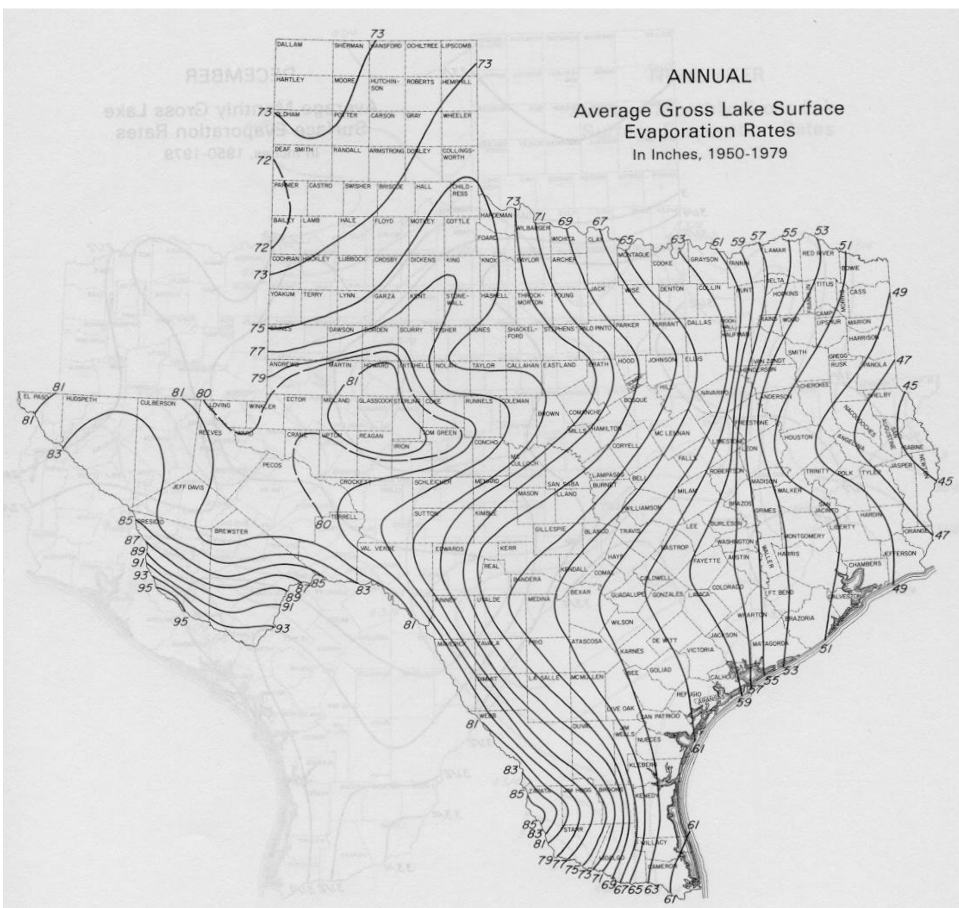


Figure 4.1 Average Annual Free Water Surface Evaporation (in inches)
(Source: NOAA, 1982)

4.3 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 1.1 of the Water Quality Technical Manual*, 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 4.1 is about 64 inches.
- For a clay loam the infiltration rate is I = 0.18 ft/day (Table 4.2).
- 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 4.4 shows summary calculations for this site for each month of the year.

Table 4.4 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

<u>Explanation of Table line number:</u>

1. Months of year
2. Days per month
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 4.2, 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 4.2 Dallas/Fort Worth Precipitation Information

4.4 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.

5.0 Rainfall Tables

5.1 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 5.1. 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 section 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.

   $$\mathbf{i = b/(t + d)^e}$$

   where:

   i = rainfall intensity (inches per hour)

   t = rainfall duration (minutes)

   b, d and e = parameters found at the top of each of the tables in Section 5.0.

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.

5.2 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.

5.3 References

[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.

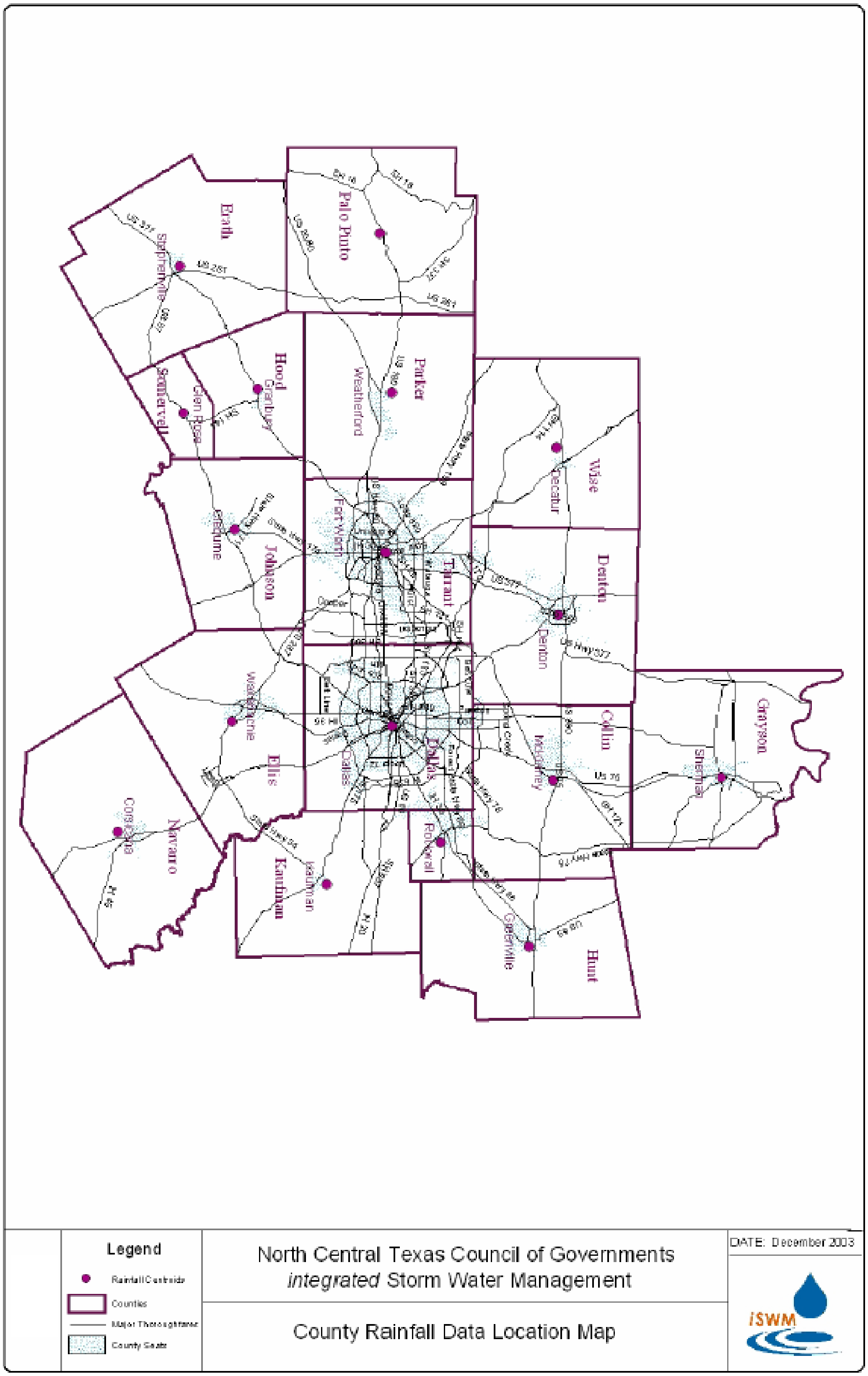


Figure 5.1 County Rainfall Data Location Map

Table 5.1 Collin County Rainfall Data

Hours	Minutes	1	2	5	10	25	50	100
		Return Period (Years)						
	Coefficients	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 5.2 Dallas County Rainfall Data

		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 5.3 Denton County Rainfall Data

Hours	Coefficients	Return Period (Years)						
		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	T8	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 5.4 Ellis County Rainfall Data

Hours	Minutes	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 5.5 Erath County Rainfall Data

		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 5.6 Hood County Rainfall Data

Hours	Coefficients / Minutes	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 5.7 Hunt County Rainfall Data

		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 5.8 Johnson County Rainfall Data

		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 5.9 Kaufman County Rainfall Data

		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 5.10 Navarro County Rainfall Data

		Return Period (Years)						
	Coefficients	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 5.11 Palo Pinto County Rainfall Data

		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 5.12 Parker County Rainfall Data

	Coefficients	Return Period (Years) 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 5.13 Rockwall County Rainfall Data

		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 5.14 Somervell County Rainfall Data

		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 5.15 Tarrant County Rainfall Data

		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 5.16 Wise County Rainfall Data

		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

Table 5.17 500-year Storm Rainfall Data

		County									County						
		Collin	Dallas	Denton	Ellis	Erath	Hood	Hunt	Johnson	Kaufman	Navarro	Palo Pinto	Parker	Rockwall	Somervell	Tarrant	Wise
	e	0.6609	0.6756	0.6819	0.7082	0.7270	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	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	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	11.51	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	11.11	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	10.74	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	10.40	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	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	9.80	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	9.52	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	9.27	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	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	8.80	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	8.59	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	8.38	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	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	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	7.84	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	7.68	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	7.52	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	7.38	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	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	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	6.97	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	6.85	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	6.73	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	6.62	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	6.51	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	6.40	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	6.30	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	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	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	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	5.93	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	5.85	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	5.77	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	5.69	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	5.61	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	5.54	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	5.47	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	5.40	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	5.33	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	5.26	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	5.20	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	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	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	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	4.96	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	4.91	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	3.06	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	2.35	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	1.41	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	0.86	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	0.52	0.59	0.54	0.59	0.60	0.51	0.52	0.59	0.53	0.54	0.54

6.0 Hydrologic Soils Data

6.1 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. An example SSURGO Soil Map for West Tarrant County is shown in Figure 6.1.

6.2 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.

6.3 Downloading Soil Surveys

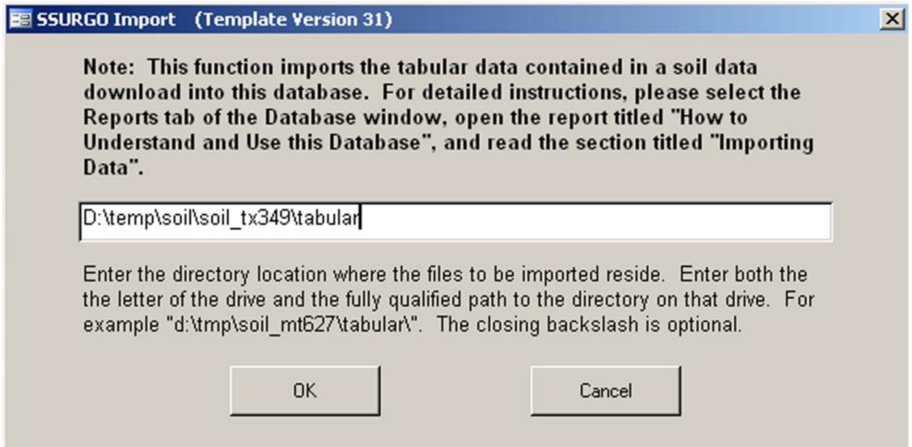
Following is a procedure for downloading data from the NRCS web site and importing it into ArcGIS.

Downloading SSURGO Soil Data into ArcInfo 9.x

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>

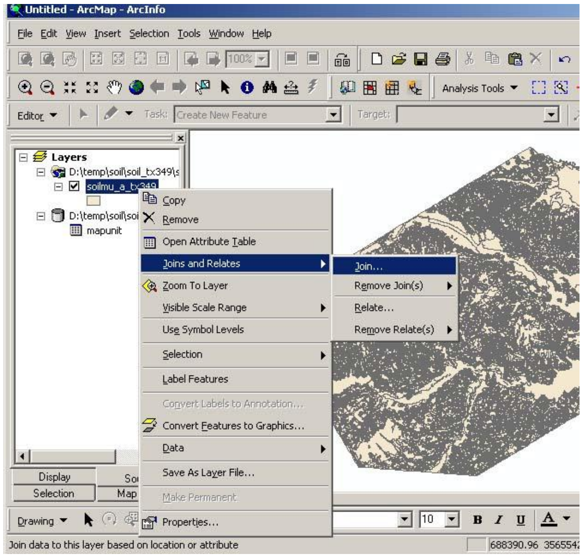
1. Extract the soildb_US_2002.zip file into the same destination folder by using the "extract to here" command in WinZip. This will extract the template database.
2. Open the template database and input the path name to the tabular data. This will build the SSURGO database and allow the creation of reports and queries.

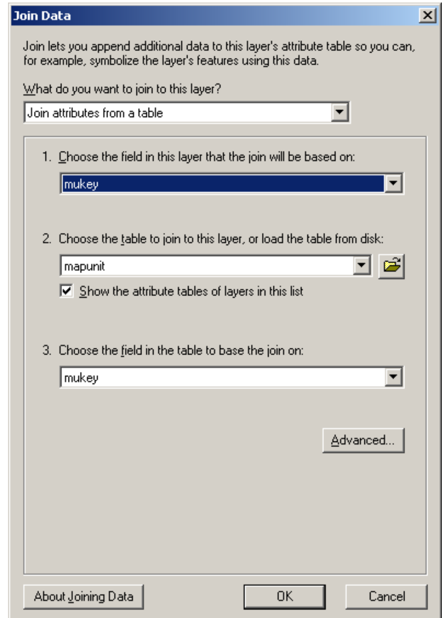


3. Once the data is imported into the database, a report can be run. With the soil reports dialog box up, press the **Select All** option and generate the report. Note: Regardless of what report you wish to run, all reports are simultaneously created. The selected report is displayed on the screen.
4. 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.

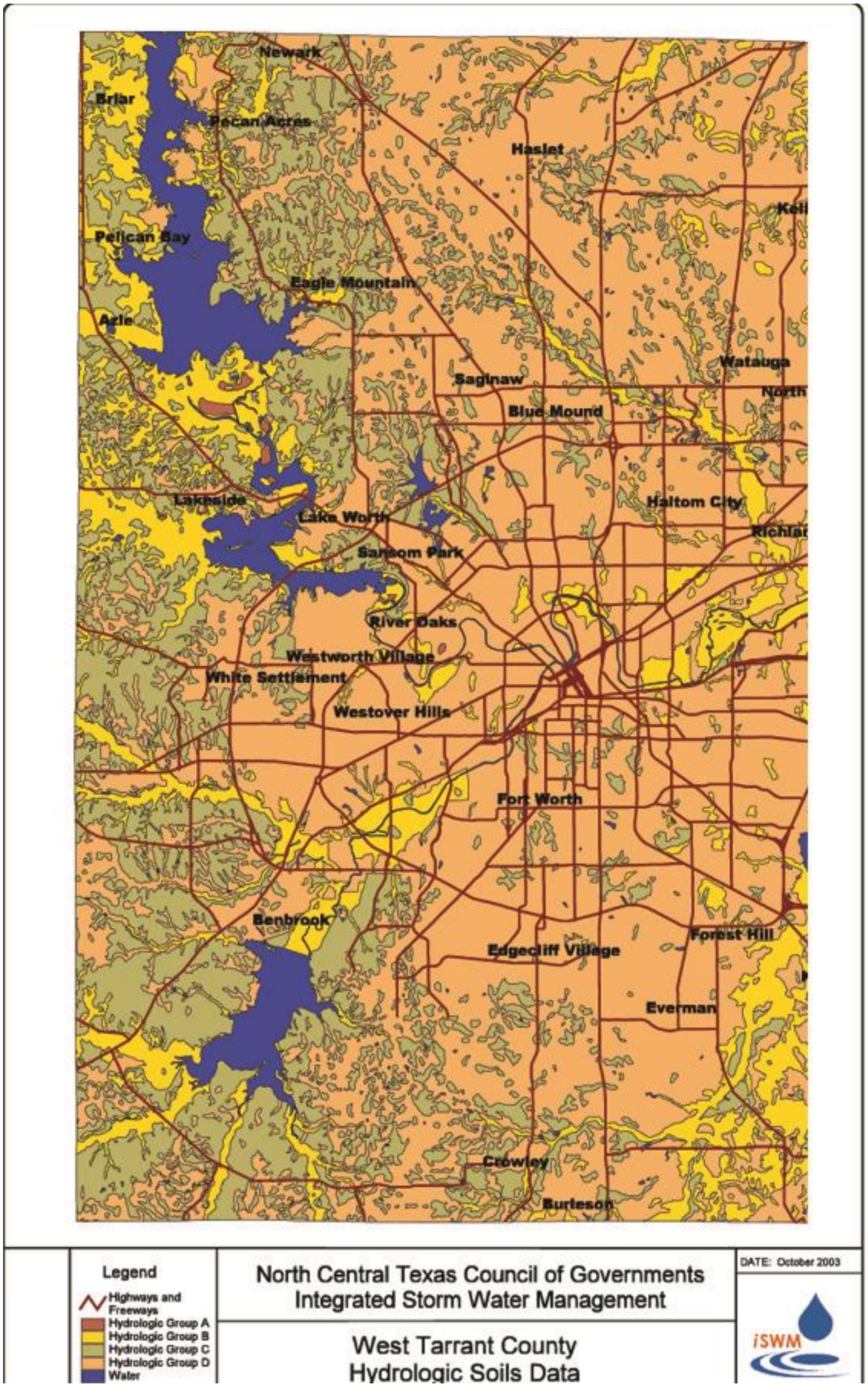


Figure 6.1 Example SSURGO Soil Map – West Tarrant County

Hydraulics:

1.0 Streets and Closed Conduits
2.0 Storage Design
3.0 Open Channels, Culverts, and Bridges
4.0 Energy Dissipation

Table of Contents

List of Tables

List of Figures

1.0 Streets and Closed Conduits

1.1 Stormwater Street and Closed Conduit Design Overview

1.1.1 Stormwater System Design

Stormwater system design is an integral component of both site and overall stormwater 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 stormwater runoff.

Stormwater collection systems must be designed to provide adequate surface drainage while at the same time meeting other stormwater management goals such as water quality, streambank protection, habitat protection, and groundwater recharge.

1.1.2 System Components

The stormwater system components consist of all the *integrated* site design practices and stormwater controls utilized on the site. Three considerations largely shape the design of the stormwater systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove stormwater 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, stormwater ponds and wetlands, and small underground pipe systems which collect stormwater runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger stormwater systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The stormwater (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 stormwater 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 section 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 2.0* covers storage design and outlet structures. *Section 3.0* covers the design of culverts, vegetated and lined open channels, and bridges. *Section 4.0* covers energy dissipation devices for outlet protection. The rest of this section covers important considerations to keep in mind in the planning and design of stormwater drainage facilities.

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:

   On-site (structural control, receiving water)

   Off-site (highway, storm drain, receiving water, regional control)

   Natural drainage system (swales)

   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 stormwater management system and plan.

1.1.4 Key Issues in Stormwater System Design

The traditional design of stormwater systems has been to collect and convey stormwater 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:

- Stormwater 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 stormwater controls to mitigate the major stormwater impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in stormwater system design.

General Design Considerations

- Stormwater 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 stormwater 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 stormwater (minor) systems.
- It is important to ensure that the combined on-site flood control system and major stormwater 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 stormwater systems and/or major stormwater structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of stormwater systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major stormwater system design capacity.

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 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.

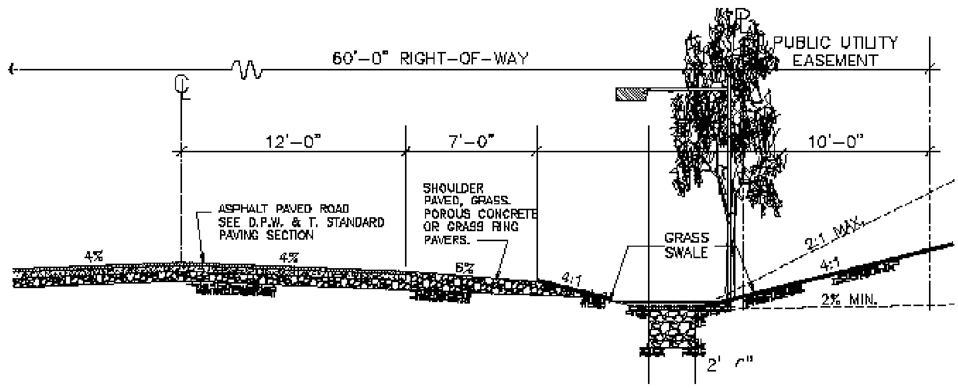


Figure 1.1 Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

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 stormwater 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:
    - Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.

- Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
- Improve aesthetics. Several smaller drains will be less obvious than one large drain.
- 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.

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 stormwater 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.

1.2 On-Site Flood Control System Design

1.2.1 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 stormwater runoff and transport it to structural control facilities, pervious areas, and/or the larger stormwater 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
- Stormwater inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in *Section 3.0.*

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.

1.2.3 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 1.1 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 1.1 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	-

1.2.4 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 stormwater 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.

Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$\mathbf{Q = [0.56/n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3}} \qquad \textbf{(1.1)}$$

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 1.1 was first presented by C.F. Izzard in 1946.

Nomograph

Figure 1.2 is a nomograph for solving Equation 1.1. Manning's n values for various pavement surfaces are presented in Table 1.2 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.

Manning's n Table

Table 1.2 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.

Uniform Cross Slope

The nomograph in Figure 1.2 is used with the following procedures to find gutter capacity for uniform cross slopes:

<u>Condition 1</u>: 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.

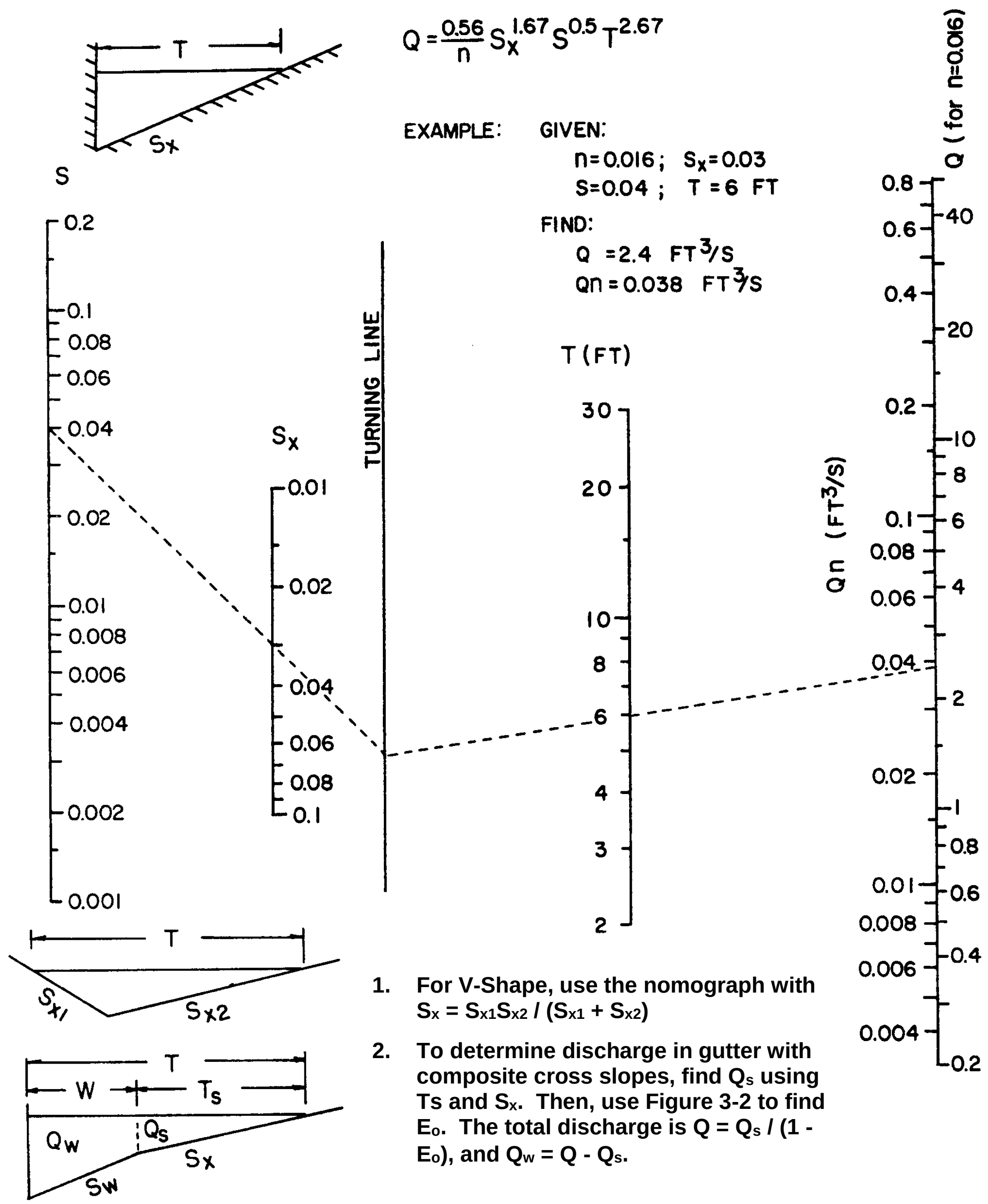
Condition 2: Find gutter flow, given spread.

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 T_s 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 1.2 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

Composite Gutter Sections

Figure 1.3 in combination with Figure 1.2 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 1.4 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 1.4 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:

$$\mathbf{Q_w = Q - Q_s} \qquad \mathbf{(1.2)}$$

Step 3 Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 1.3 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 1.2.

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 1.2 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 1.3, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \mathbf{(1.3)}$$

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 1.2.

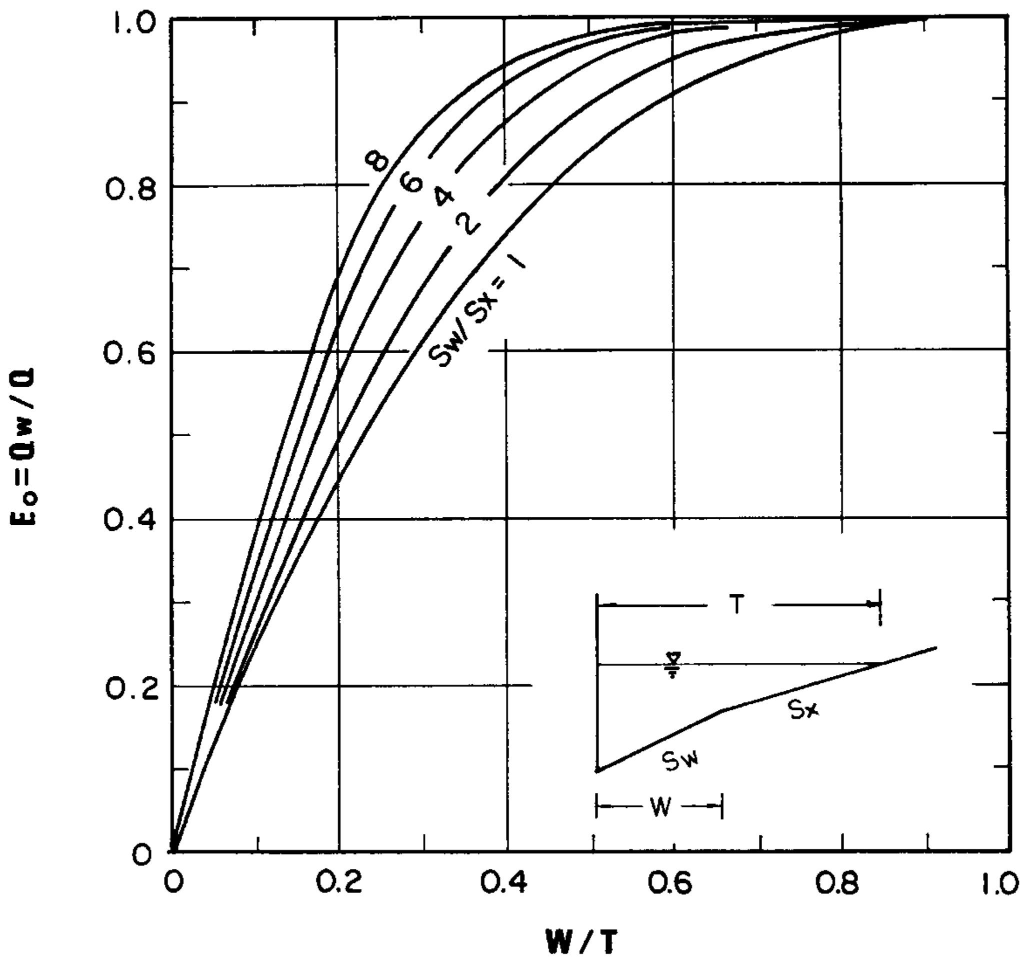


Figure 1.3 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

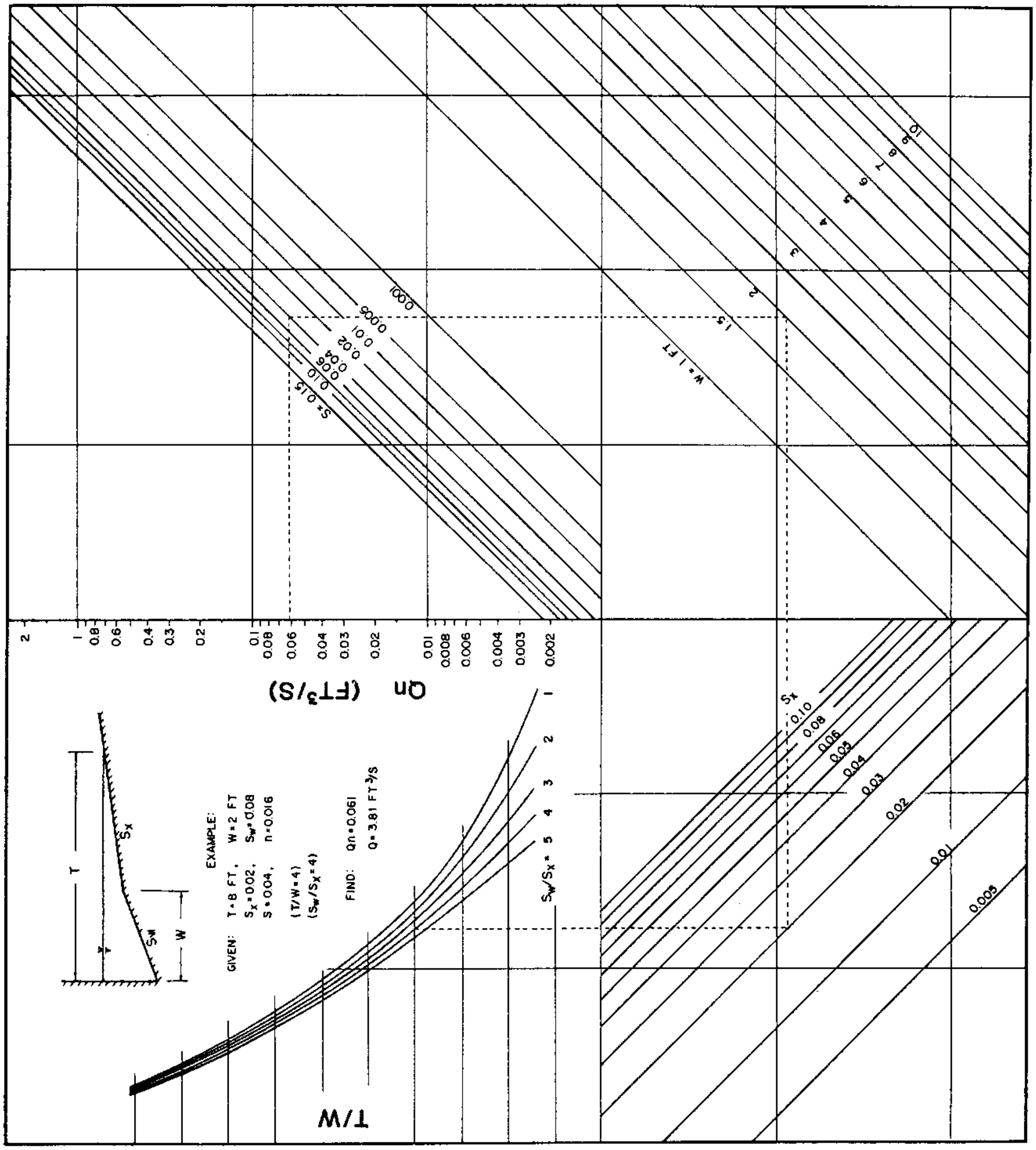


Figure 1.4 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

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 1.2, Qn = 0.03
$Q = Qn/n = 0.03/0.015 = 2.0$ cfs

b. $T = 8 - 2 = 6$ ft
$(Qn)_2 = 0.014$ (Figure 1.2) (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 1.2 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 1.3 to find $E_o = 0.64$
3. Calculate the gutter flow using Equation 1.3
$Q = 2.7/(1 - 0.64) = 7.5$ cfs
4. Calculate the gutter flow in width, W, using Equation 1.2
$Q_W = 7.5 - 2.7 = 4.8$ cfs

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:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 \qquad (1.4)$$

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 1.3 for different street widths

Table 1.3 Coefficients for Equation 1.4, 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:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3 (CS) \qquad (1.5)$$

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 1.4 and 1.5

The K values in Equation 1.5 are found in Tables 1.4 and 1.5 for different street widths. Table 1.4 is used when calculating the higher gutter flows, and Table 1.5 is used when calculating the lower gutter flows.

Table 1.4 Coefficients for Equation 1.5, 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 1.5 Coefficients for Equation 1.5, 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:

$$\text{Crown Height (feet)} = 0.5 * W * S_x$$

where,

W = street width, feet
S_x = road cross slope, ft/ft

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 1.3, 1.4, and 1.5, see HEC-22 Urban Design Manual by the Federal Highway Administration, dated August 2001.

1.2.5 Stormwater 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 *Section 1.2.6*, curb inlet design in *Section 1.2.7*, and combination inlets in *Section 1.2.8*.

1.2.6 Grate Inlet Design

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 1.6 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:

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \qquad (1.6)$$

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 1.6 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 1.3 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:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \qquad (1.7)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\,(V - V_0) \qquad (1.8)$$

where:

V = velocity of flow in the gutter, ft/s (using Q from Figure 1.2)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 1.5)

This ratio is equivalent to frontal flow interception efficiency. Figure 1.5 provides a solution of Equation 1.8, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 1.5 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:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \qquad (1.9)$$

where:

L = length of the grate, ft

Figure 1.5 provides a solution to Equation 1.9.

The efficiency, E, of a grate is expressed as:

$$E = R_fE_o + R_s(1 - E_o) \qquad (1.10)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)] \qquad (1.11)$$

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 1.5 for Curved Vane Grate, R_f = 1.0

From Figure 1.5 for Reticuline Grate, R_f = 1.0

From Figure 1.6 R_s = 0.1 for both grates

From Equation 1.11:

Q_i = 3.0[1.0 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.

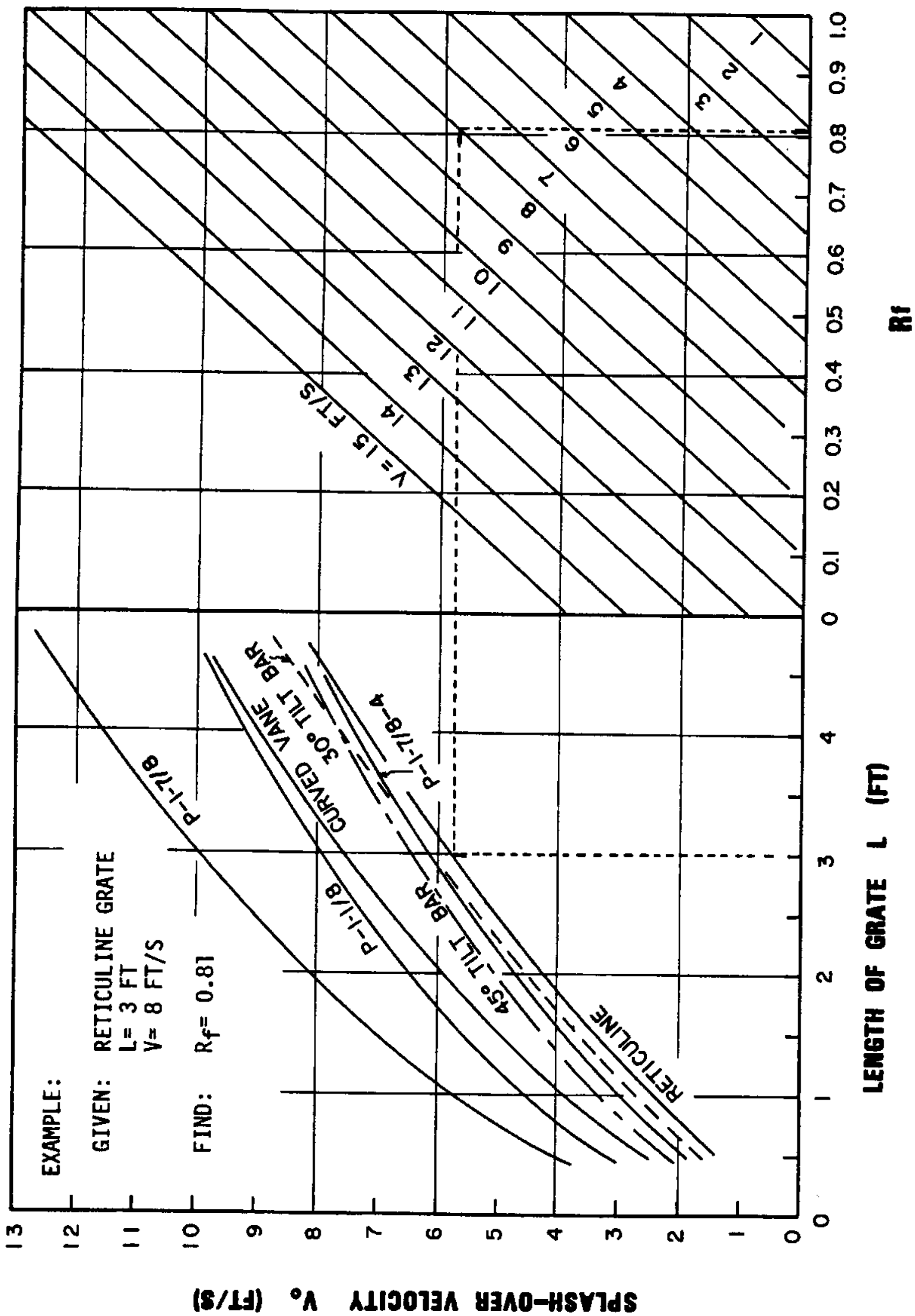


Figure 1.5 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

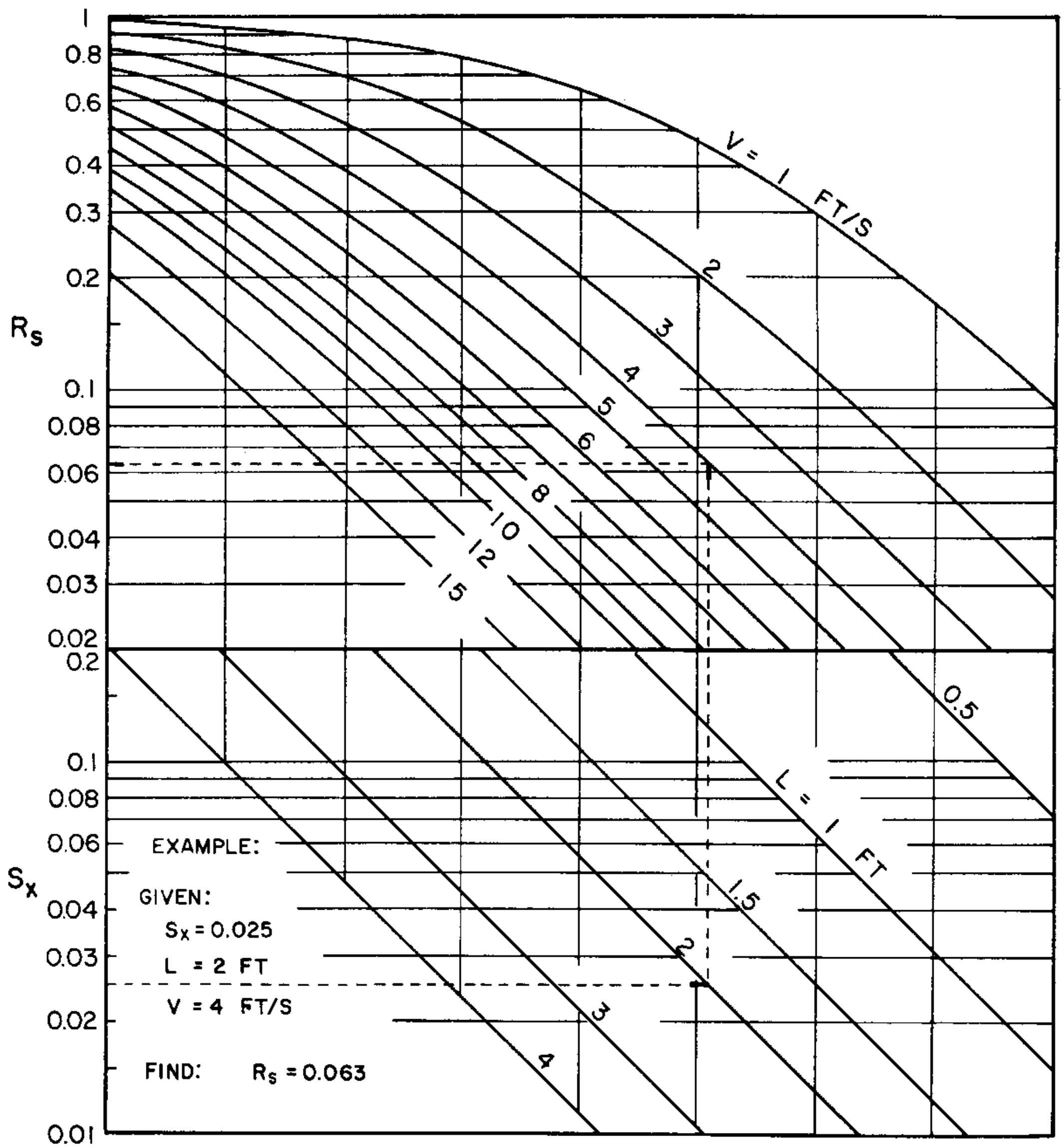


Figure 1.6 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

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:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \mathbf{(1.12)}$$

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:

$$\mathbf{Q_i = CA(2gd)^{0.5}} \qquad \mathbf{(1.13)}$$

where:

- C = 0.67 orifice coefficient
- A = clear opening area of the grate, ft^2
- g = 32.2 ft/s^2

Figure 1.7 is a plot of Equations 1.12 and 1.13 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

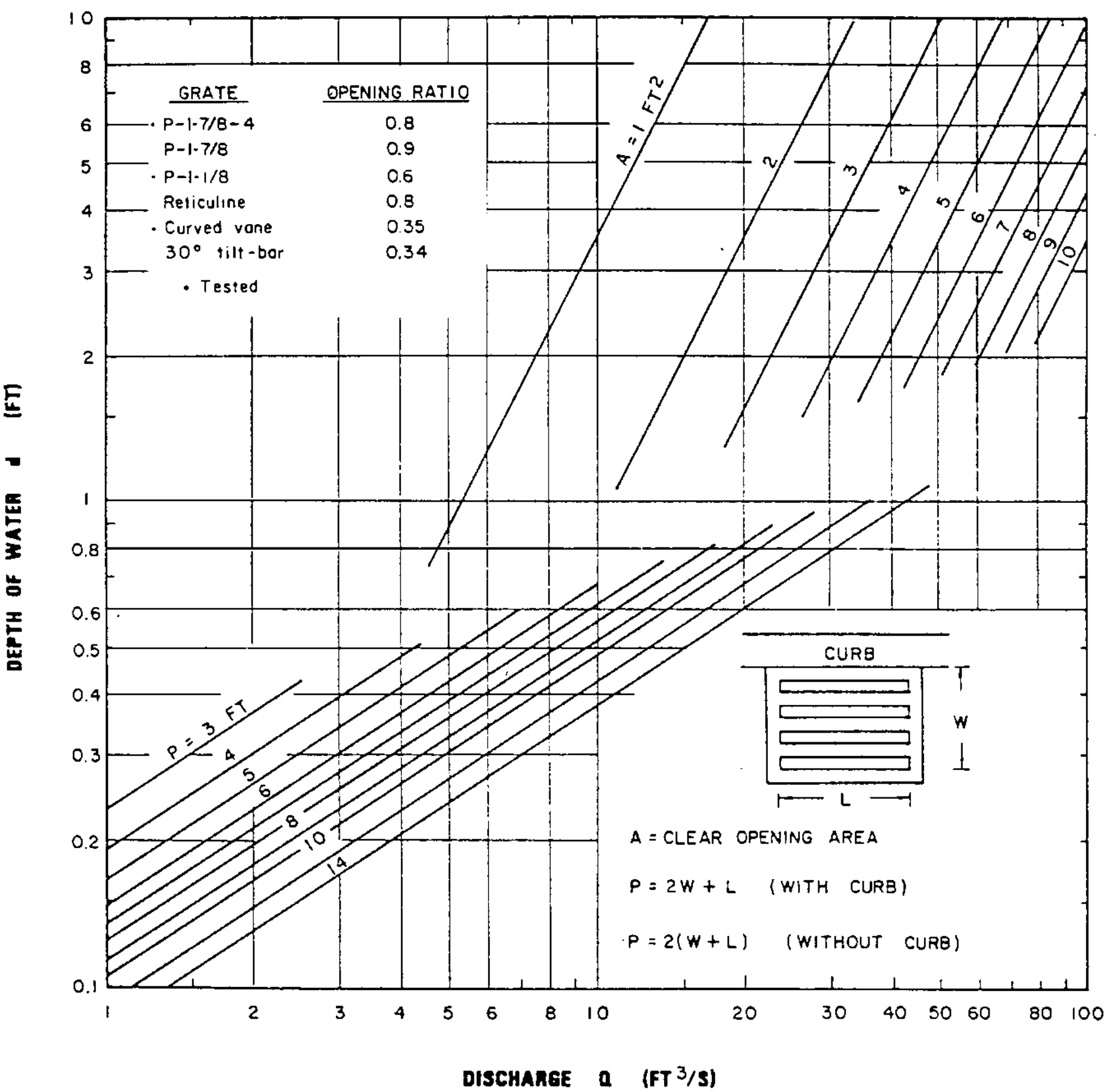
Find:

Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

Solution:

From Figure 1.7, 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 1.7 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 1.7)

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 1.2)

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.

1.2.7 Curb Inlet Design

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 1.8. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 1.9.

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:

$$S_e = S_x + S'_w E_o \qquad \textbf{(1.14)}$$

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 (ft/ft)

S'_w = (a/12W)

where:

a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 1.8 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.

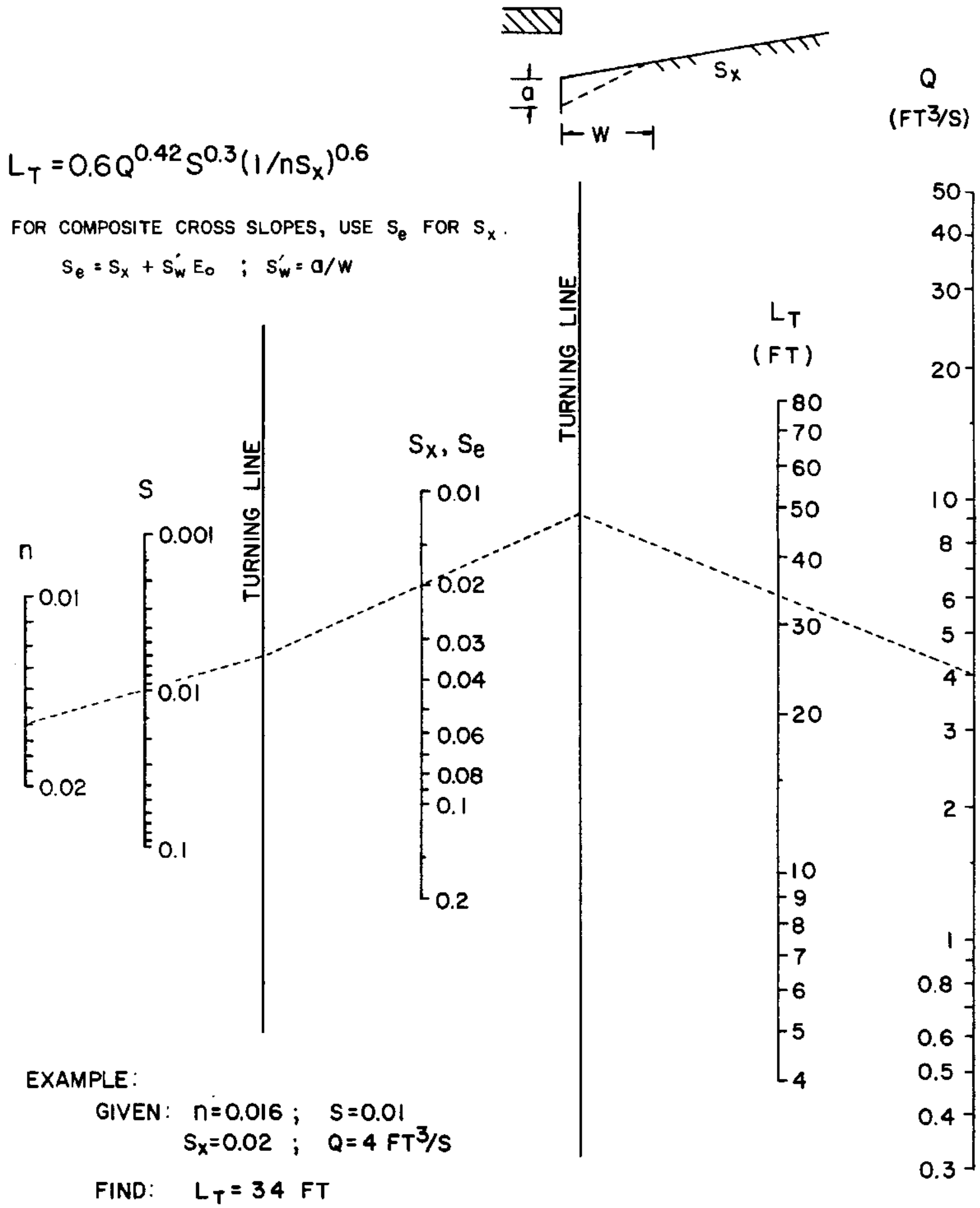


Figure 1.8 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

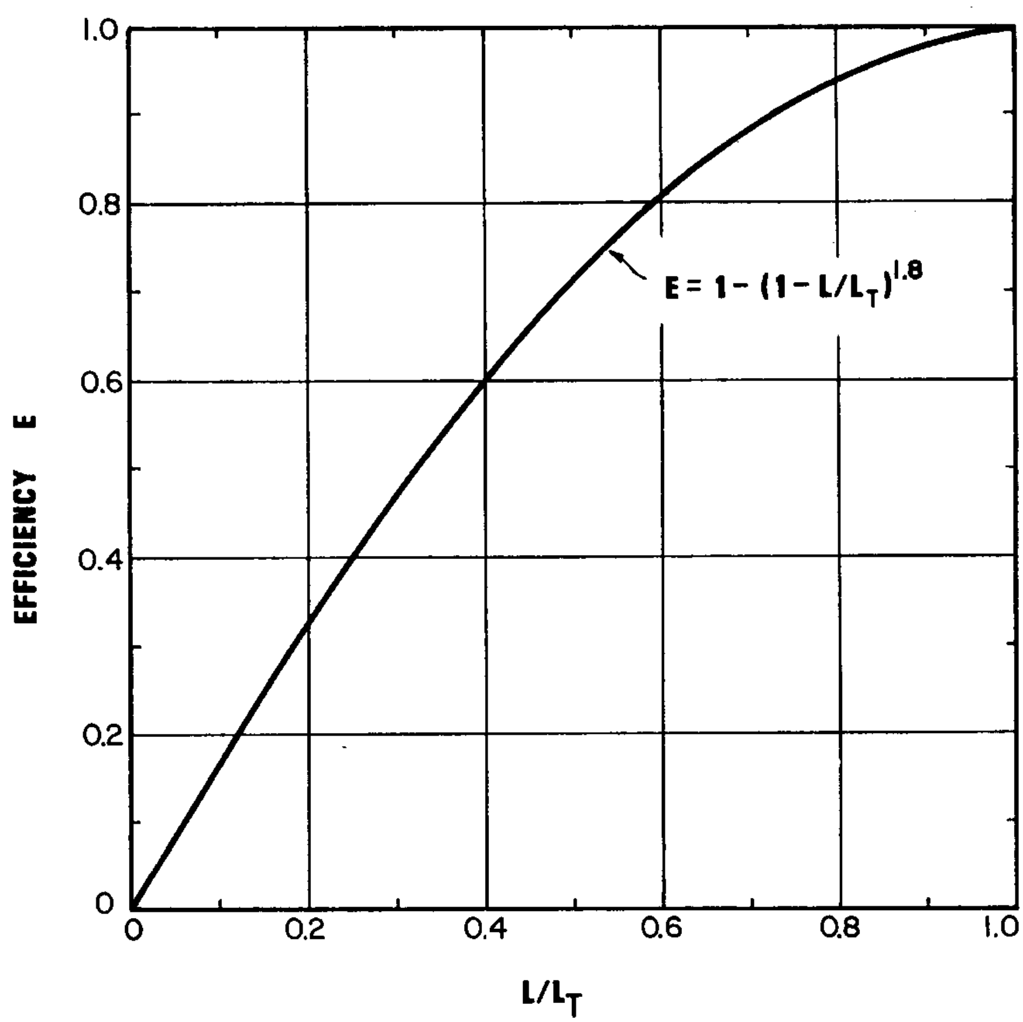


Figure 1.9 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

<u>Design Steps</u>

Steps for using Figures 1.8 and 1.9 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 1.2

Step 2 Enter Figure 1.8 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 1.9 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 1.2)

Solution:

1. From Figure 1.8, L_T = 41 ft, L/L_T = 10/41 = 0.24
   From Figure 1.9, 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 1.4)
   T = 3.5 x 2 = 7 ft
   W/T = 2/7 = 0.29 ft
   E_o = 0.72 (from Figure 1.3)
   Therefore, $S_e = S_x + S'_wE_o$ = 0.03 + 0.083(0.72) = 0.09
   From Figure 1.8, L_T = 23 ft, L/L_T = 10/23 = 0.43
   From Figure 1.9, 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.

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 1.10, 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 1.10). The weir portion of Figure 1.10 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 1.11. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 1.12.

Design Steps

Steps for using Figures 1.10, 1.11, and 1.12 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 1.2
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 (1.10, 1.11, or 1.12 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.

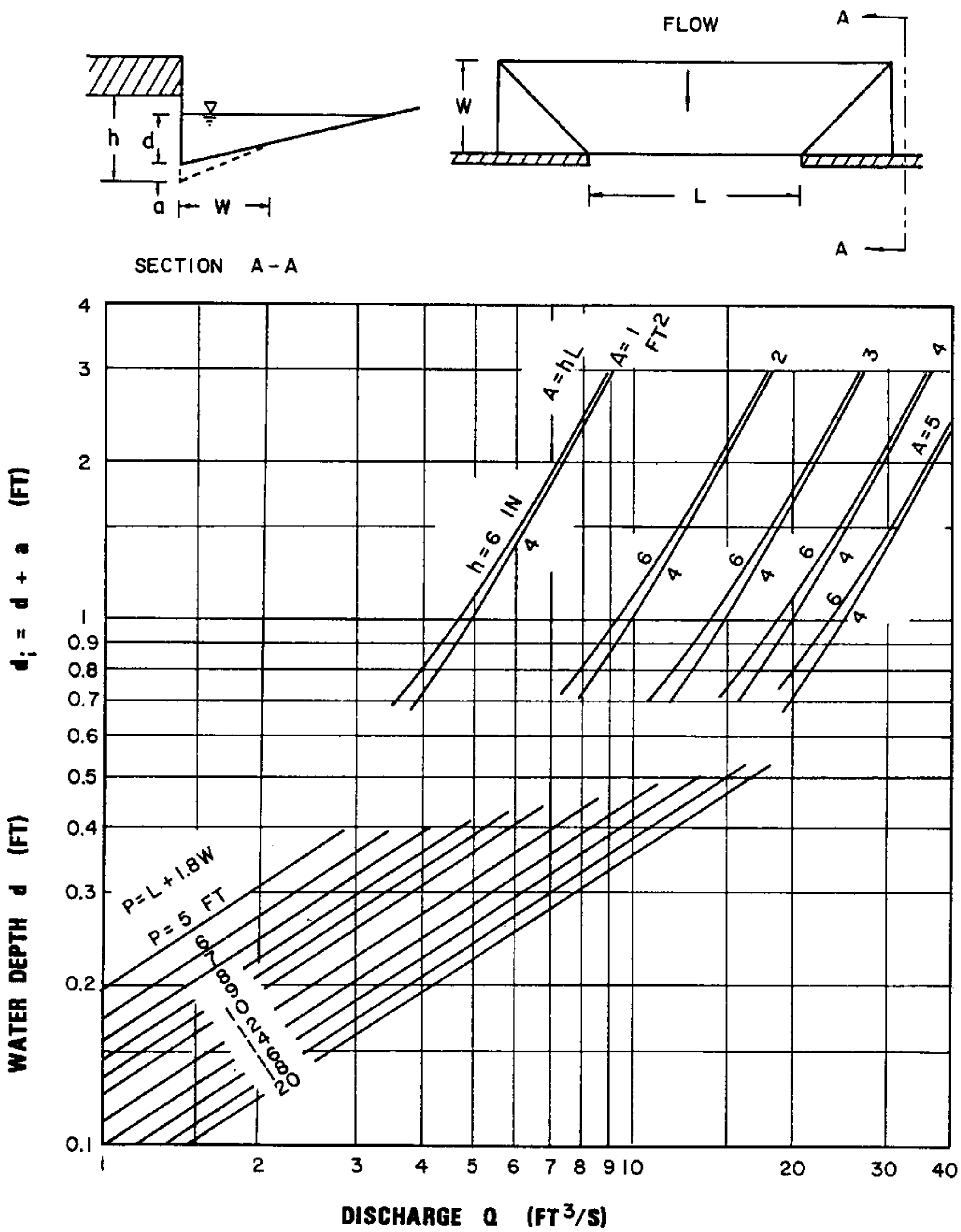


Figure 1.10 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

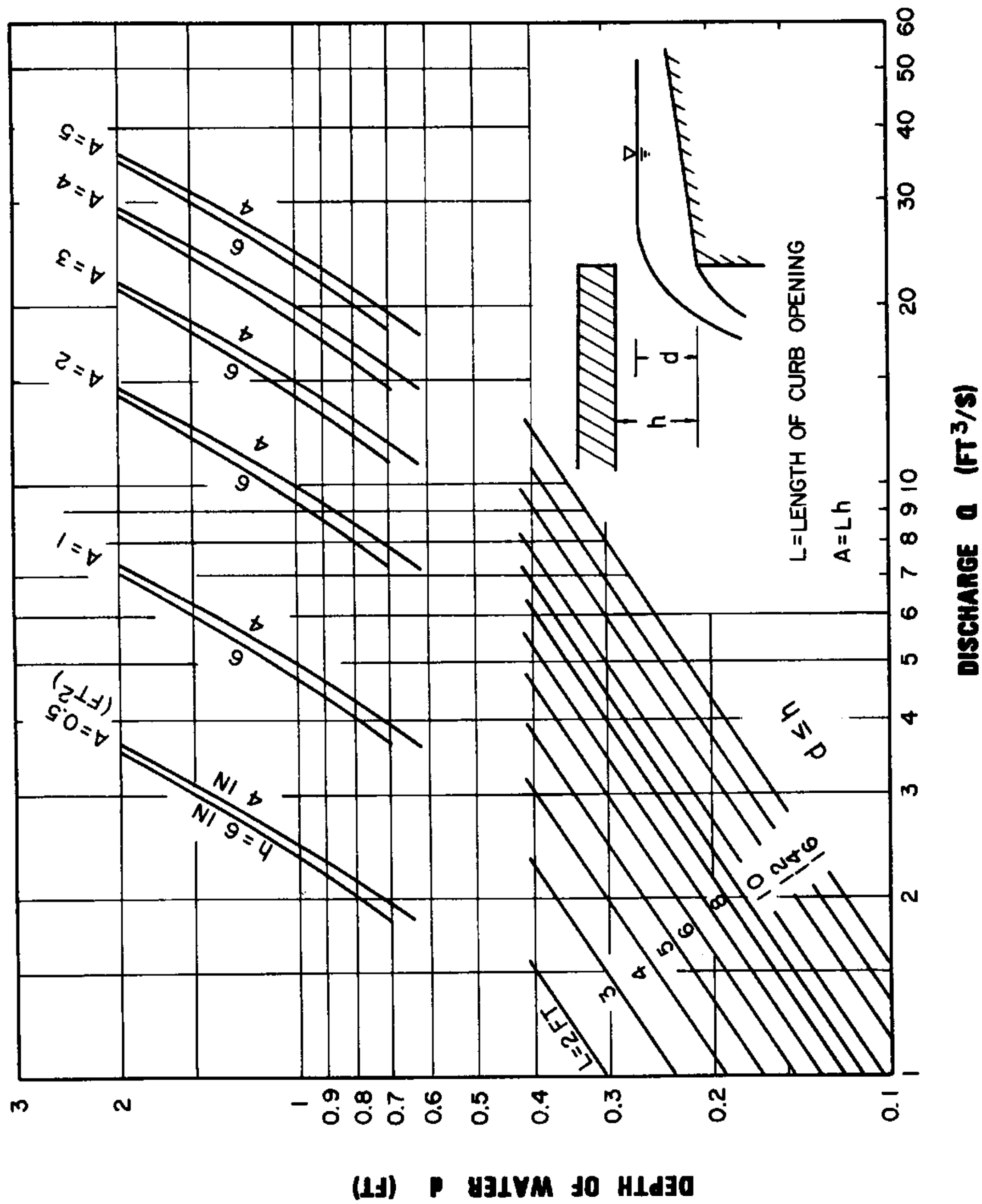


Figure 1.11 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

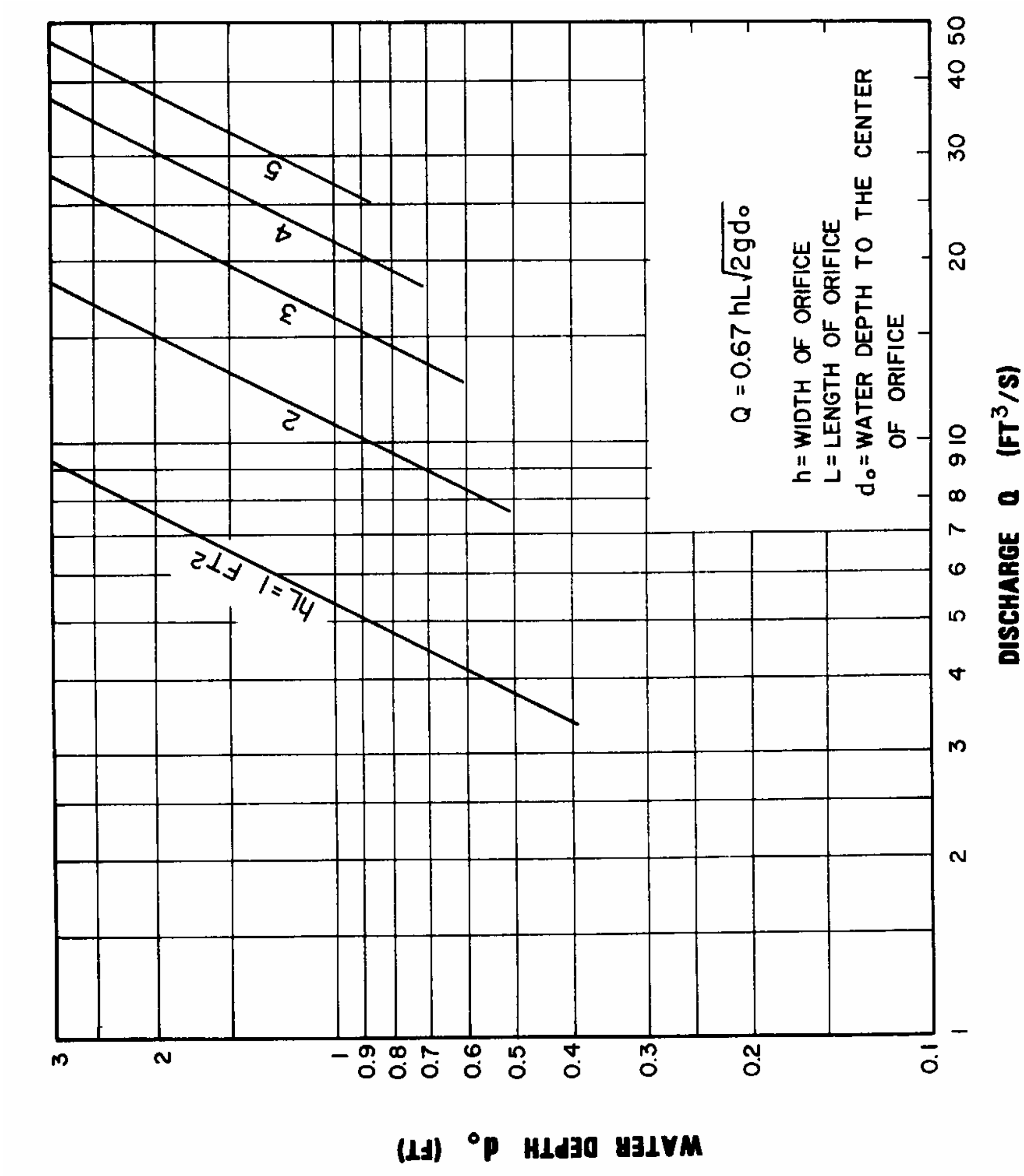


Figure 1.12 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

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 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 1.10 is applicable (lower portion of the figure).

   P = L + 1.8W = 5 + 3.6 = 8.6 ft
   From Figure 1.9, Q_i = 5 cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

1.2.8 Combination Inlets

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 1.5, 1.6, and 1.7.

Combination Inlets in Sump

All debris carried by stormwater 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 1.10, 1.11, and 1.12 for curb-opening inlets should be used for design.

1.2.9 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 stormwater 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 this section.

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 1.13 and 1.14).

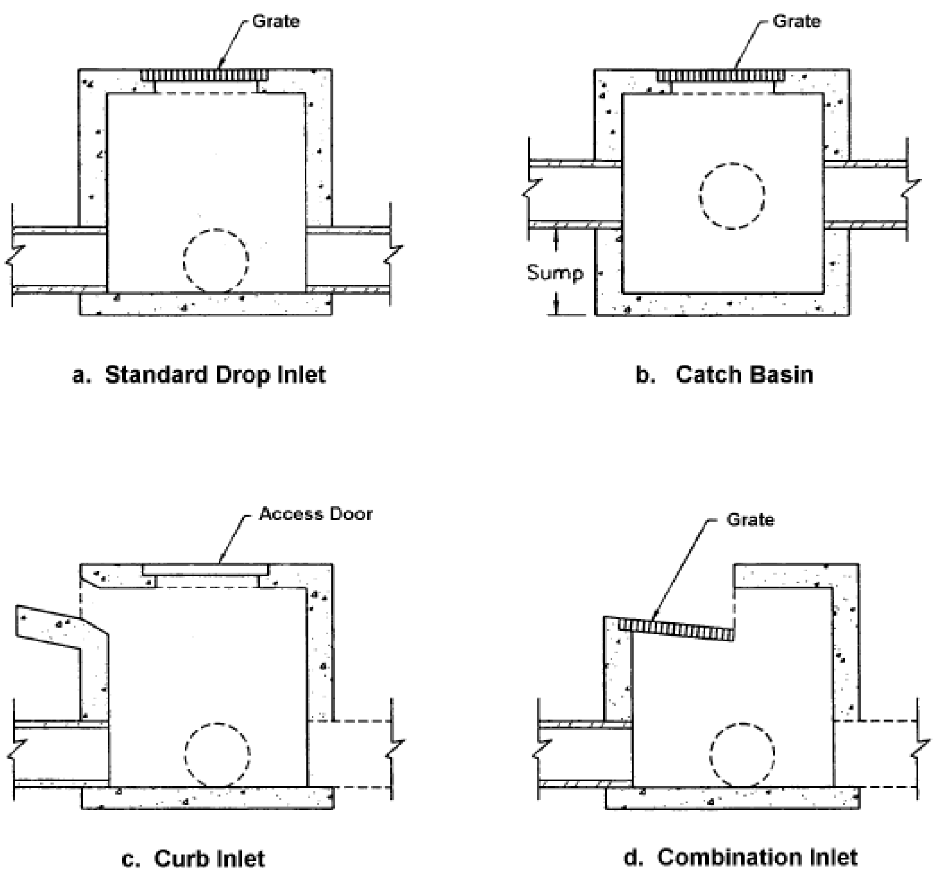


Figure 1.13 Inlet Structures
(HEC 22, 2001)

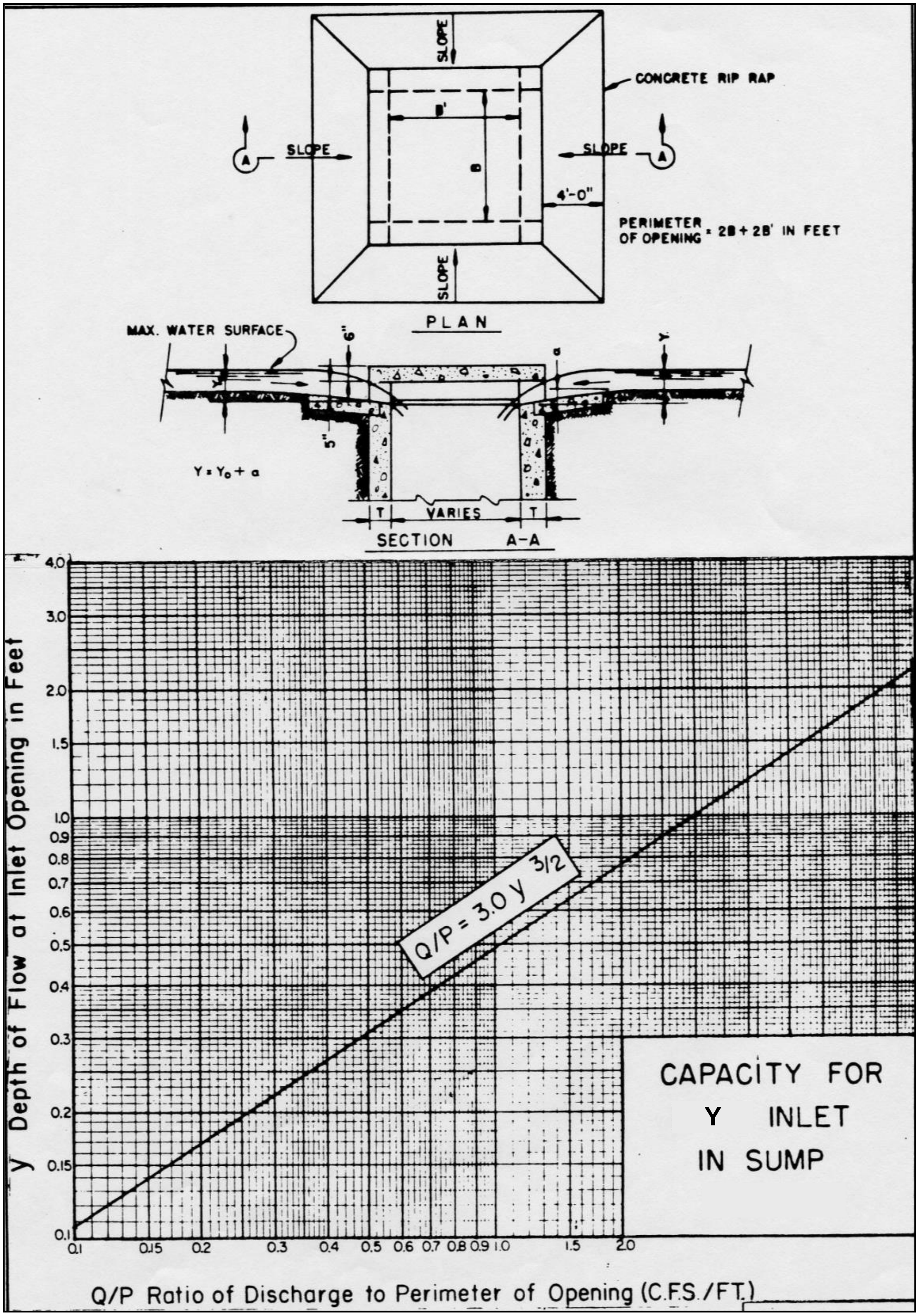


Figure 1.14 Capacity for Y Inlet in Sump
(Fort Worth, 1967)

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).

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 1.15 and 1.16 (HEC 22, 2001). The materials commonly used for access hole construction are precast concrete and cast-in-place concrete.

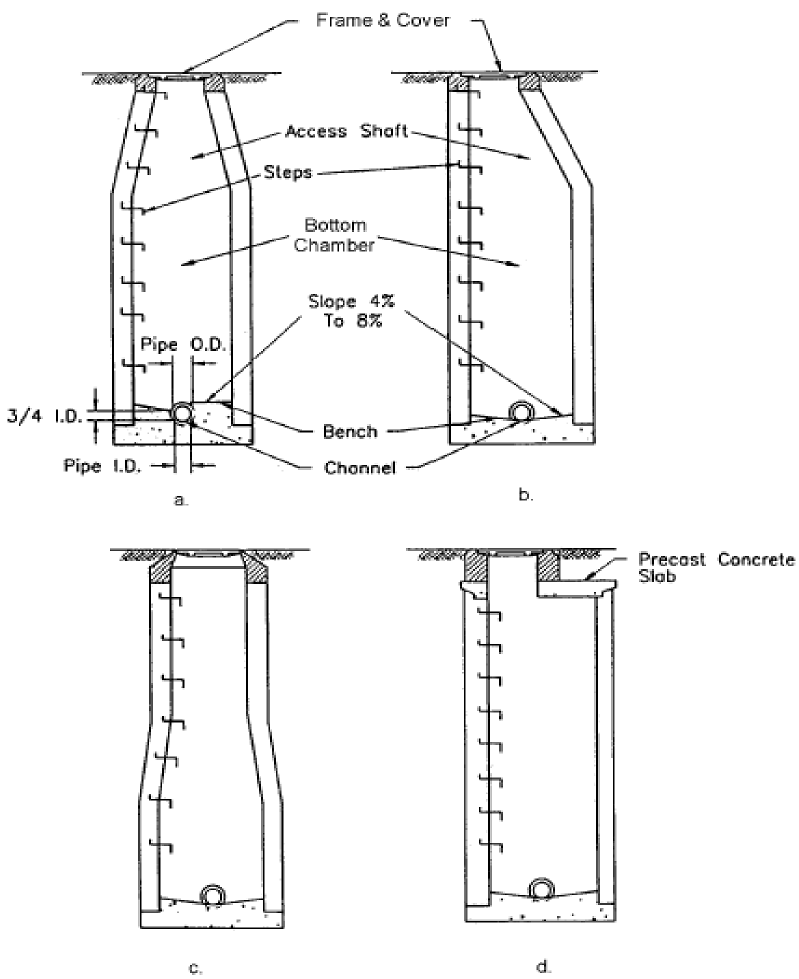


Figure 1.15 Typical Access Hole Configurations.
(HEC22, 2001)

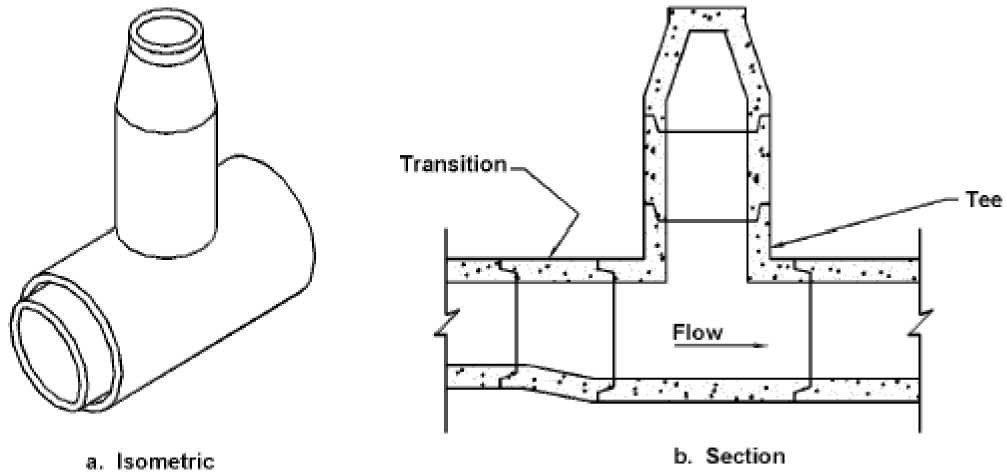


Figure 1.16 "Tee" Access Hole for Large Storm Drains
(HEC 22, 2001)

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 3.9 of the Criteria Manual*.

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 1.31) 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 and is explained in *Section 1.2 of the Hydrology Technical Manual*. 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 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. 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 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.

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:

$$\mathbf{V = (1.486/n)\ R^{2/3}\ S^{1/2}} \qquad \textbf{(1.15)}$$

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:

$$\mathbf{Q = (1.486/n)\ A\ R^{2/3}\ S^{1/2}} \qquad \textbf{(1.16)}$$

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:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \qquad \textbf{(1.17)}$$

$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \qquad \textbf{(1.18)}$$

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:

$$H_f = [0.453\ n^2V^2L]/[R^{4/3}] \qquad (1.19)$$

$$H_f = [(2.87\ n^2V^2L]/[D^{4/3}] \qquad (1.20)$$

$$H_f = [(185n^2(V^2/2g)L]/[D^{4/3}] \qquad (1.21)$$

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 1.17. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 1.8. 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.

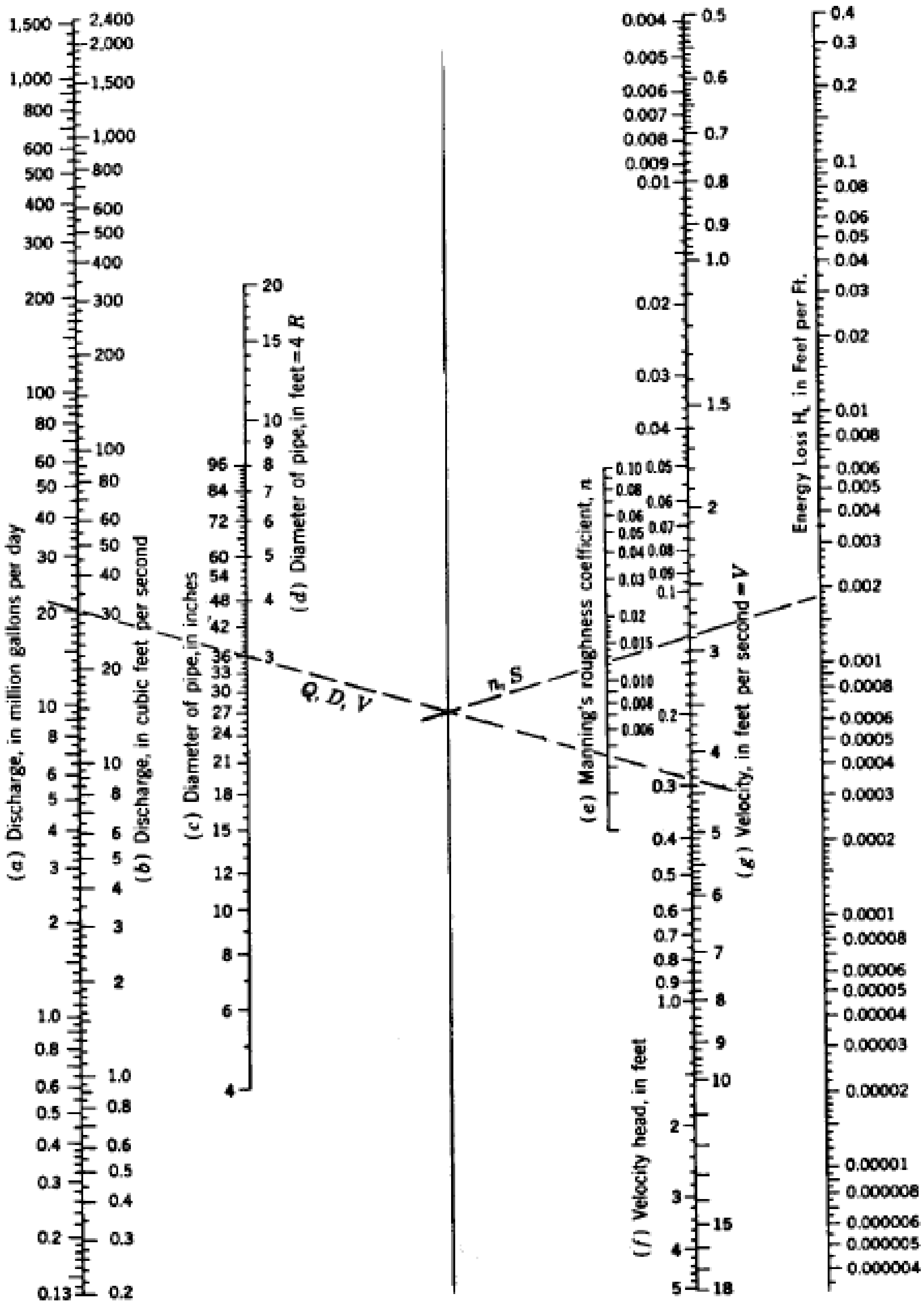
Figure 1.18 illustrates storm drain capacity sensitivity to the parameters in the Manning's Equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased 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 1.19a and 1.19b 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 1.7 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 1.7 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**



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Figure 1.17 Solution of Manning's Equation for Flow in Storm Drains-English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Institute)

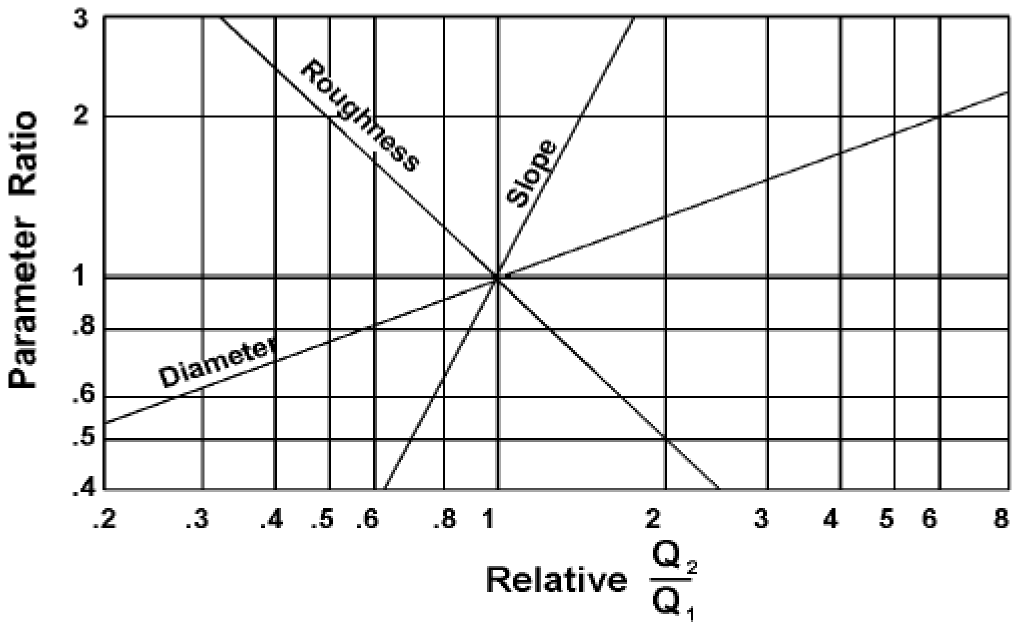


Figure 1.18 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

Table 1.8 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.**

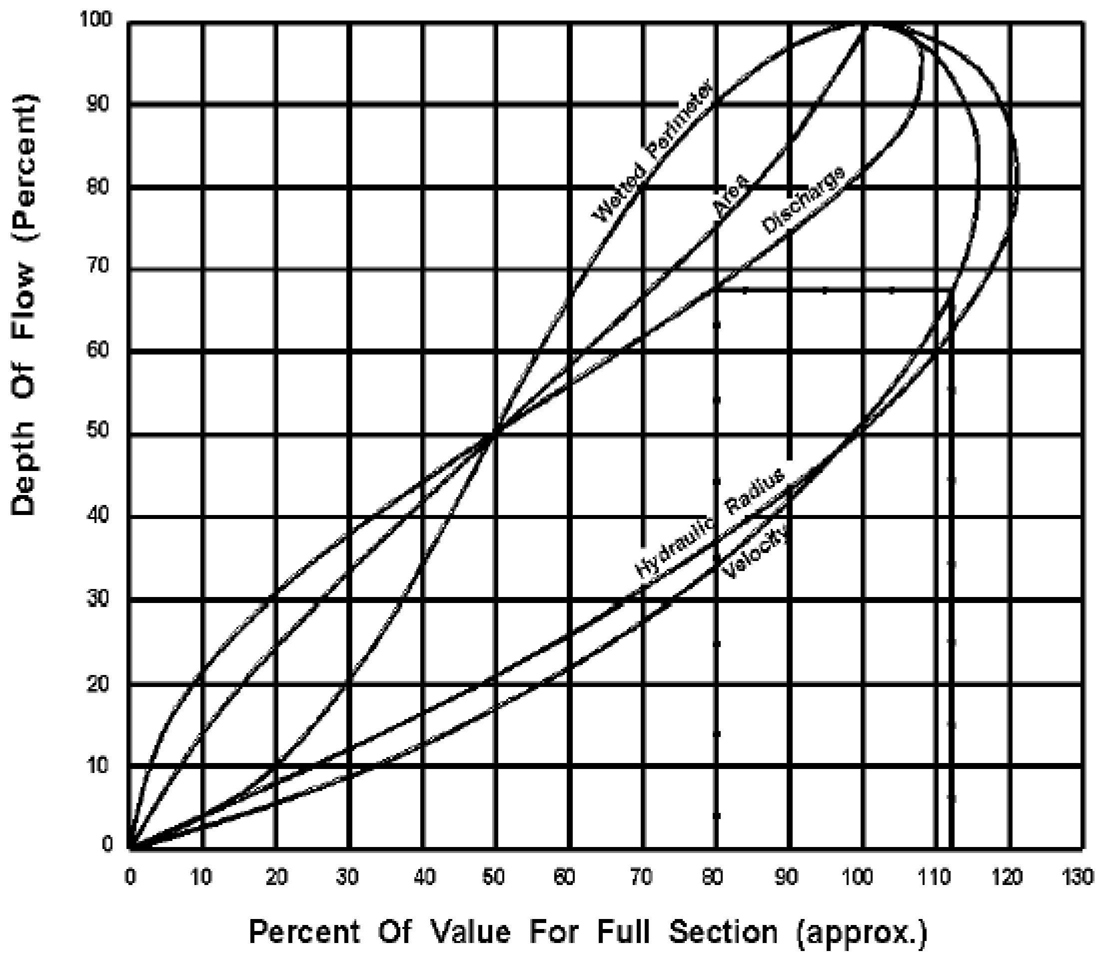


Figure 1.19a Hydraulic Elements of Circular Section
(HEC 22, 2001)

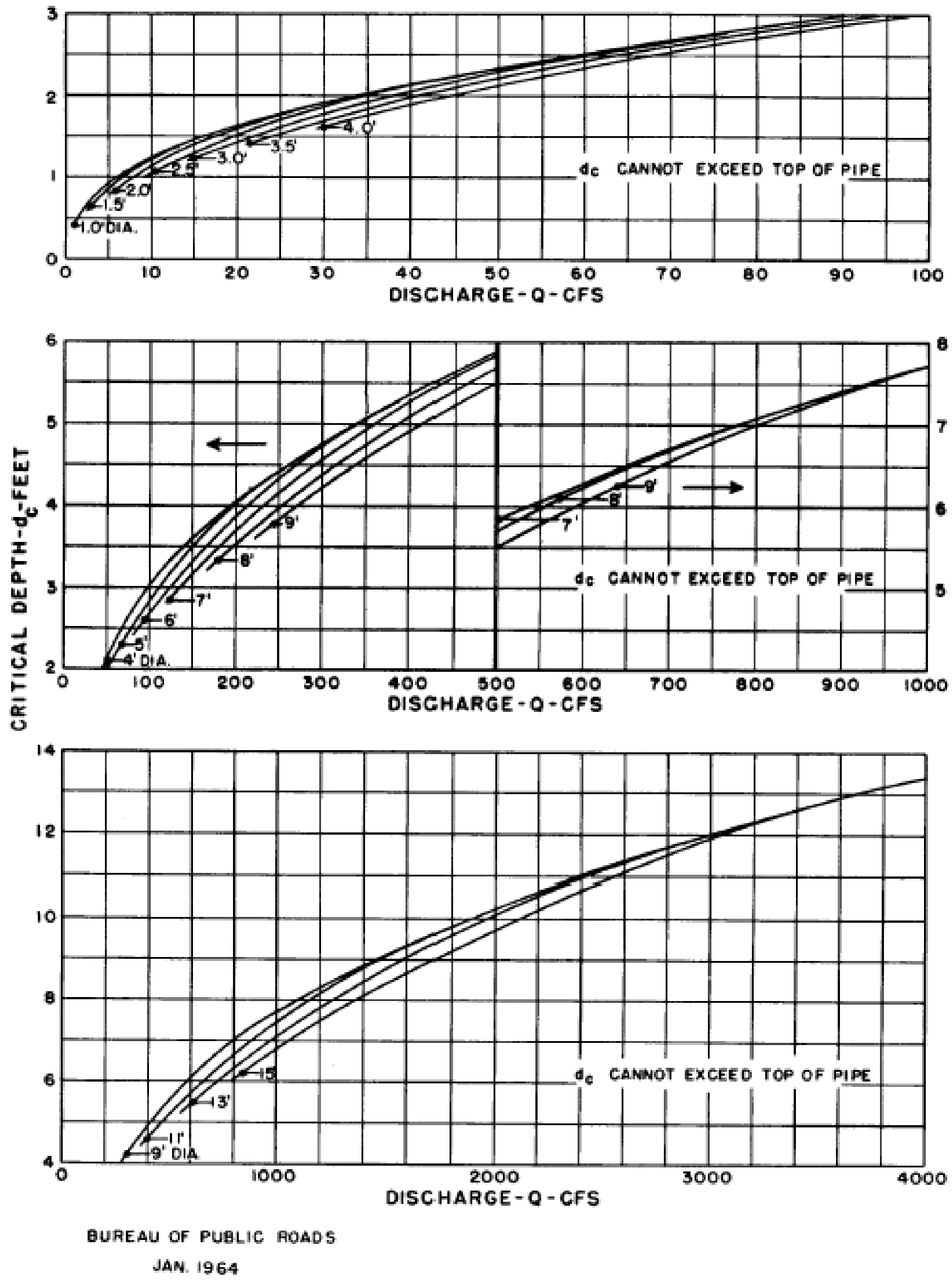


Figure 1.19b Critical Depth in Circular Pipe-English Units
(HEC 22, 2001)

Minimum Grades and Desirable Velocities

The minimum slopes are calculated by the modified Manning's formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \qquad (1.22)$$

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 1.22 may be expressed as:

$$S = 2.87(nV)^2/D^{4/3} \qquad (1.23)$$

For a minimum velocity of 2.5 fps, the minimum slope equation becomes:

$$S = 17.938(n^2/D^{4/3}) \qquad (1.24)$$

where:

D = diameter, ft

Table 1.9 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.

<u>Minimum Grades</u>

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 1.9 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.

Table 1.9 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

Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural stormwater 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 2.0* for more information).

1.2.10 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.

$$\mathbf{E = V^2/2g + p/\gamma + z} \qquad \mathbf{(1.25)}$$

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:

$$V_1^2/2g + p_1/\gamma + z_1 = V_2^2/2g + p_2/\gamma + z_2 - H_f - \Sigma H_m \qquad (1.26)$$

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 1.20 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.

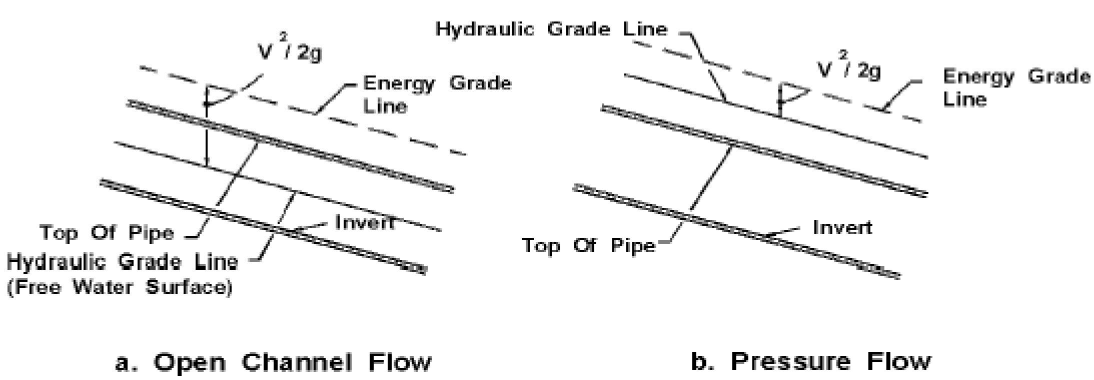


Figure 1.20 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

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 1.2.11*.

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 stormwater. 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 1.10 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and flood mitigation 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 1.10 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 1.10 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
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 4.0* 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.

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 presents 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 1.2.12*.

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:

$$\mathbf{H_f = S_f\,L} \qquad \mathbf{(1.27)}$$

where:

H_f = friction loss, ft

S_f = friction slope, ft/ft

L = length of pipe, ft

The friction slope in Equation 1.27 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by Equation 1.27, 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 1.20.

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:

$$\mathbf{H_o = 1.0\,[(V_o^2/2g) - (V_d^2/2g)]} \qquad \mathbf{(1.28)}$$

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.

Bend Losses

The bend loss coefficient for storm drain design is minor but can be estimated using the following formula (AASHTO, 1991):

$$h_b = 0.0033\ (\Delta)\ (V^2/2g) \qquad (1.29)$$

where:

Δ = angle of curvature in degrees

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 1.16 and 1.21.

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 1.30 and 1.31 respectively.

$$H_c = K_c\ [V_1^2/(2g) - V_2^2/(2g)] \qquad (1.30)$$

$$H_e = K_e\ [V_1^2/(2g) - V_2^2/(2g)] \qquad (1.31)$$

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)

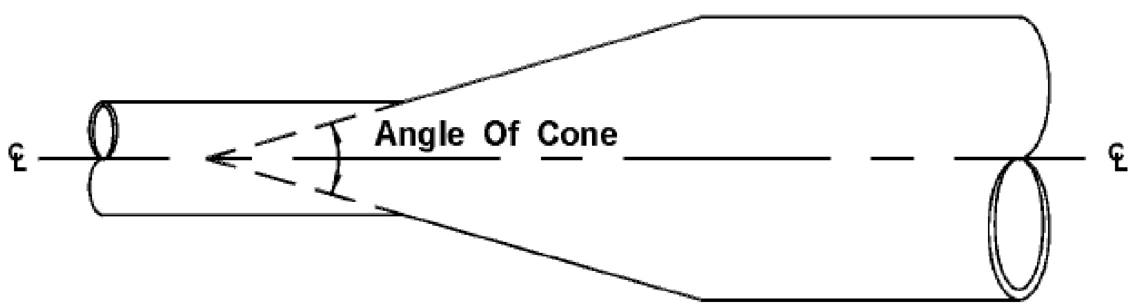


Figure 1.21 Angle of Cone for Pipe Diameter Changes

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 1.11a. Typical values of K_c for sudden contractions are tabulated in Table 1.11b. The angle of the cone that forms the transition is defined in Figure 1.21.

Table 1.11a Typical Values for K_e for Gradual Enlargement of Pipes in Non-Pressure Flow

D_2/D_1	Angle of Cone						
	10º	20º	45º	60º	90º	120º	180º
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 1.11b 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 1.12 and 1.13 can be used with Equation 1.32 for sudden and gradual expansions respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 1.14 can be used in conjunction with Equation 1.33 (ASCE, 1992).

$$H_e=K_e\,(V_1^2/2g) \qquad (1.32)$$

$$H_c=K_c\,(V_2^2/2g) \qquad (1.33)$$

where:

K_e = expansion coefficient (Tables 1.13 and 1.14)

K_c = contraction coefficient (Table 1.15)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

g = acceleration due to gravity 32.2 ft/s^2

Table 1.12 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 1.13 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 1.14 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_2, 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 1.14 Values of K_e for Determining Loss of Head due to Sudden Contraction													
D_2/D_1	Velocity, V_2, 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_2 = velocity in smaller pipe (downstream of transition)
(ASCE, 1992)

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:

$$H_j=[((Q_o\ V_o) - (Q_i\ V_i) - (Q_l\ V_l\cos\theta)) / (0.5g(A_o+A_i))] + h_i - h_o \qquad (1.34)$$

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 1.22)

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 later in this section 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 1.35. Applicable coefficients (K_{ah}) are tabulated in Table 1.15. 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.

$$H_{ah}=K_{ah}\,(V_o^2/2g) \qquad (1.35)$$

Table 1.15 Head Loss Coefficients
(FHA, Revised 1993)

Structure Configuration	K_{ah}
Inlet-straight run	0.5
Inlet-angled through	
90º	1.5
60º	1.25
45º	1.1
22.5º	0.7
Manhole-straight run	0.15
Manhole-angled through	
90º	1
60º	0.85
45º	0.75
22.5º	0.45

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.

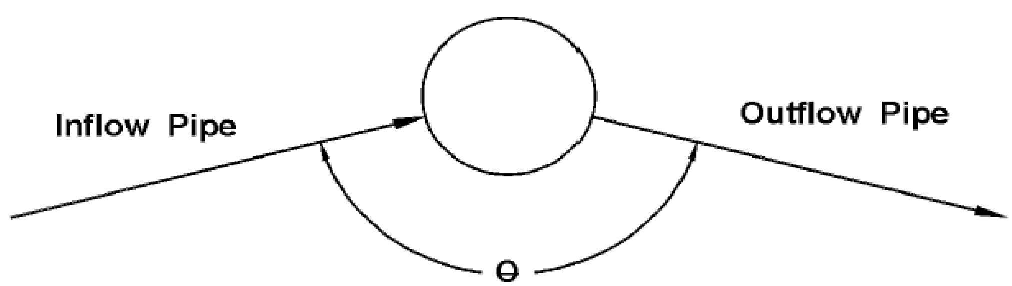


Figure 1.22 Head Loss Coefficients

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 1.36. Experimental studies have determined that the K value can be approximated by the relationship in Equation 1.37 when the inflow pipe invert is below the water level in the access hole.

$$\mathbf{H_{ah} = K\,(V_o^2/2g)} \qquad \textbf{(1.36)}$$

$$\mathbf{K = K_O\,C_D\,C_d\,C_Q\,C_p\,C_B} \qquad \textbf{(1.37)}$$

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 1.36 to K_e as reported in Table 1.16. 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 3.31a).

The initial head loss coefficient, K_o in Equation 1.38, 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 1.6. This deflection angle is represented in Figure 1.22.

$$\mathbf{K_o = 0.1\,(b/D_o)(1\text{-}\sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta} \qquad \textbf{(1.38)}$$

where:

- θ = angle between the inflow and outflow pipes (Figure 1.22)
- 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 1.39. Otherwise C_D is set equal to 1.

$$\mathbf{C_D=(D_o/D_i)^3} \qquad \textbf{(1.39)}$$

where:

- D_o = outgoing pipe diameter
- D_i = inflowing pipe diameter

Table 1.16 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, 33.7º or 45º 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, 33.7º or 45º 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 30º to 75º 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 10º to 25º 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)

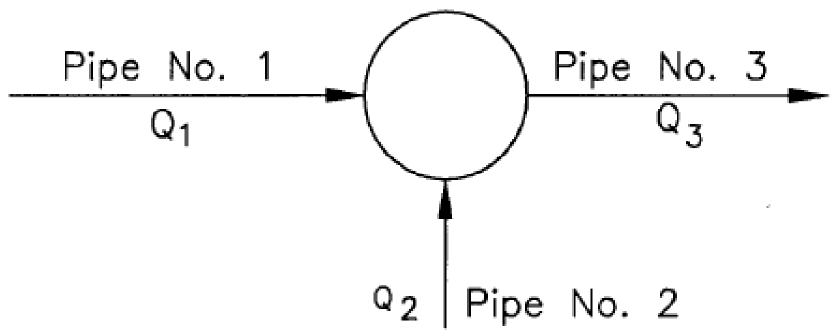


Figure 1.23 Relative flow effect

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 1.40.

$$\mathbf{C_D = 0.5(d_{aho}/D_o)^{0.6}} \qquad \mathbf{(1.40)}$$

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 1.41. 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.

$$\mathbf{C_Q = (1- 2\sin\theta)\,[1-(Q_i / Q_o)]^{0.75} + 1} \qquad \mathbf{(1.41)}$$

where:

C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (Figure 1.22)

Q_i = flow in the inflow pipe

Q_o = flow in the outflow pipe

As can be seen from Equation 1.41, 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 1.23 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 1.41,
$C_Q = (1 - 2 \sin\theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 3/4)^{0.75} + 1$
$C_Q = 1.35$

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 1.41,
$C_Q = (1 - 2 \sin\theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 1/4)^{0.75} + 1$
$C_Q = 1.81$

The correction factor for **plunging flow**, C_p, is calculated using Equation 1.42. 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 1.23, 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 1.42.

$$C_p = 1 + 0.2(h/D_o)\,[(h - d_{aho})/D_o] \qquad (1.42)$$

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 1.17. Figure 1.24 illustrates benching methods listed in Table 1.17. 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 1.17 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, $d_{aho}/D_o \leq 1.0$

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.

Composite Energy Loss Method

The Energy Loss Method described in earlier in the section 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.

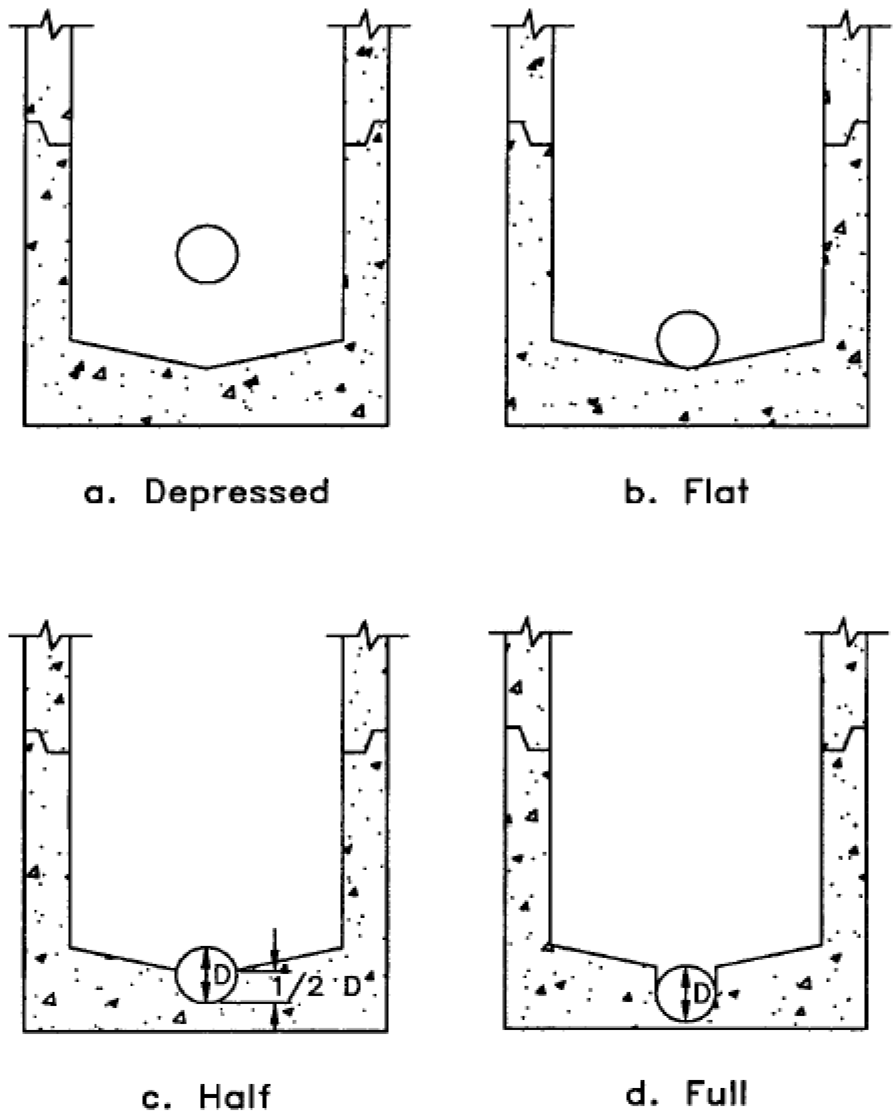


Figure 1.24 Access to Benching Methods

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 1.25. 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.

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 1.2.4 of the Hydrology Technical Manual* 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 this section 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 1.18 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 1.17 and 1.18. If the pipe is not flowing full, the velocity can be determined from Figure 1.19a.

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 1.33. 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 this section.

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 1.25 Preliminary Storm Drain Computation Sheet

1.2.11 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 1.26 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 1.27 and Figure 1.28 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 1.27) is used to calculate the HGL and EGL elevations while Table B (Figure 1.28) 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.

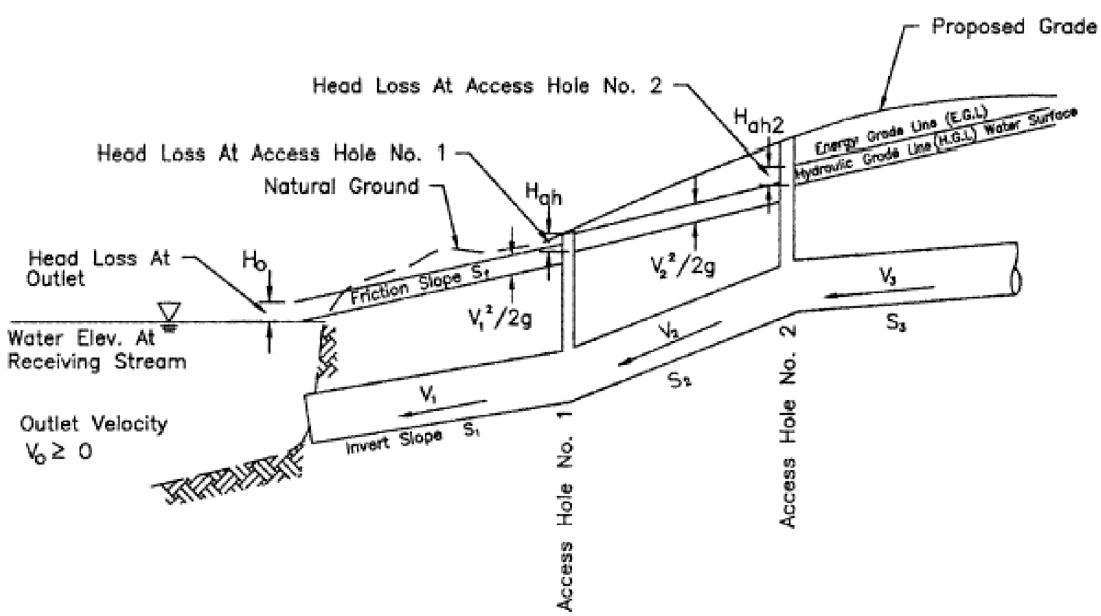


Figure 1.26 Energy and Hydraulic Grade Line Illustration

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 1.27 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 1.28 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 1.19b. 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 1.19a 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 1.19b. 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 1.19a and critical depth using Figure 1.19b. 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 1.19 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 1.29 through 1.34 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 1.36 and 1.37. 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 1.35 to K_e as reported in Table 1.15. 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 1.39 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 1.40 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 1.41, 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 1.42, 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 1.17. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 1.37, 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.

1.2.12 Storm Drain Design Example

The following storm drain design example illustrates the application of the design procedures outlined in *Section 1.2.10*.

Example of Preliminary Storm Drain Design

Given: The roadway plan and section illustrated in Figure 1.29, duration intensity information in Table 1.19 and inlet drainage area information in Table 1.18. 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 1.2.10* determine appropriate pipe sizes and inverts for the system illustrated in Figure 1.29.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in *Section 1.2.10*.

Solution:

(1) Preliminary Storm Drain Design

Step 1. Figure 1.29 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 1.30 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2. Drainage areas, runoff coefficients, and times of concentration are tabulated in Figure 1.31. Example problems documenting the computation of these values are included in this section

Starting at the upstream end of a conduit run, Steps 3 and 4 from *Section 1.2.10* are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 1.31. The column by column computations for each section of conduit follow:

Table 1.18 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 1.19 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

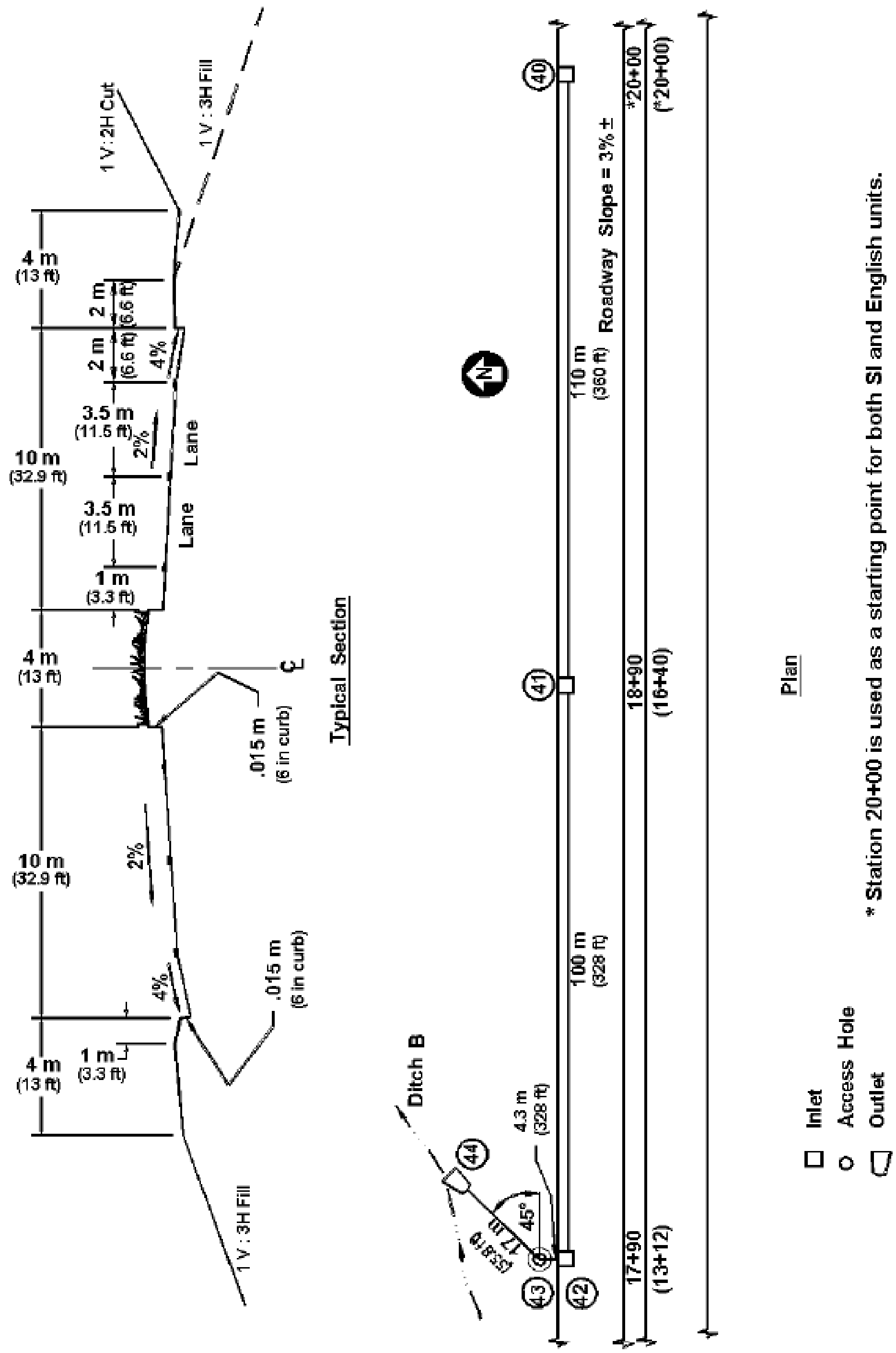


Figure 1.29 Roadway Plan and Sections for Example

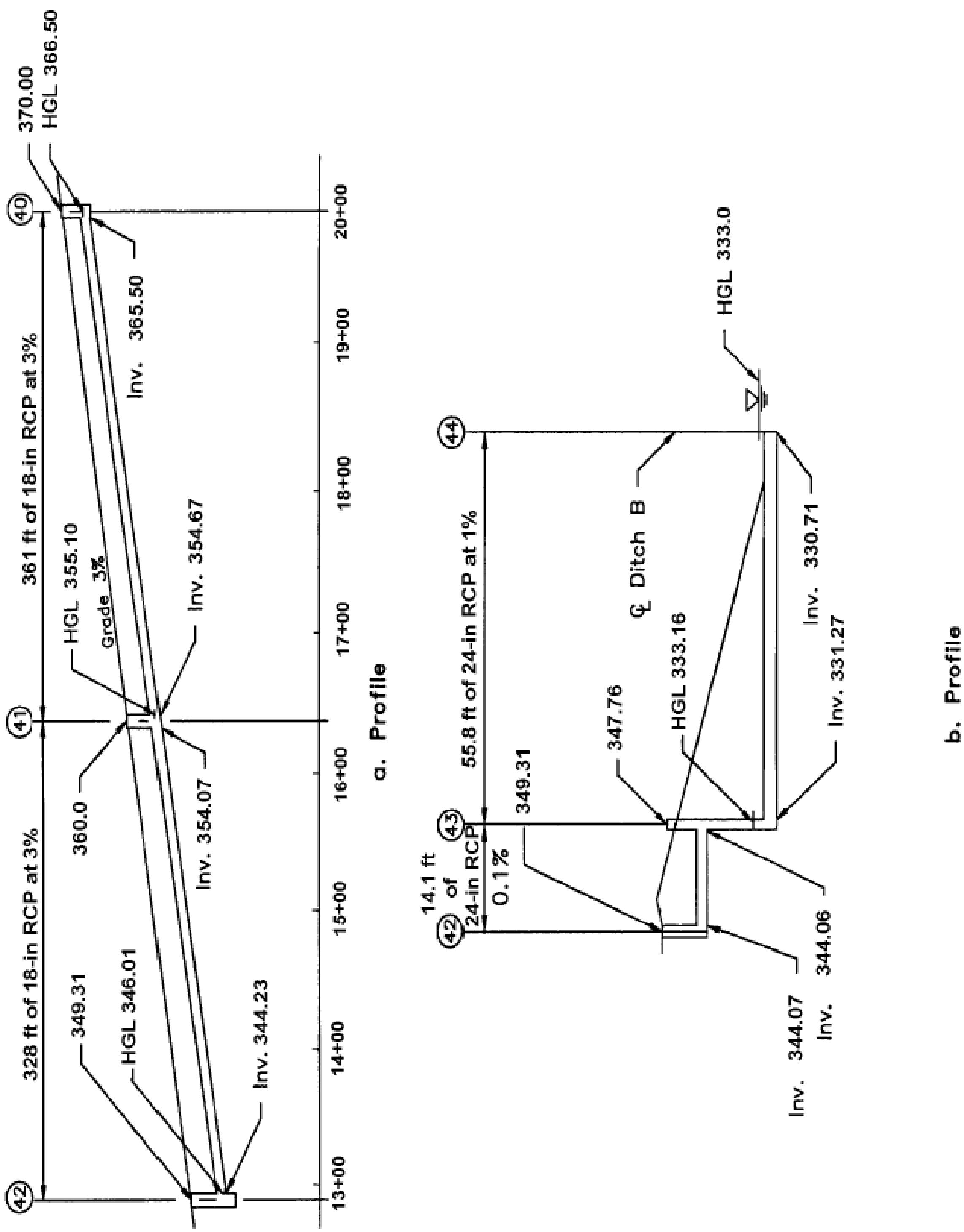


Figure 1.30 Storm Drain Profiles for Example

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)	(ft³ /s)	(ft)	(ft³/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 1.31 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 1.30
Col. 4 Inlet Area	A_i = 0.64 ac	Table 1.18
Col. 5 Total Area	A_t = 0.64 ac	Total area up to inlet 40
Col. 6 "C"	C = 0.73	Table 1.18
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	Table 1.18
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 1.19; System time less than 5 minutes therefore, use 5 minutes
Col. 12 Runoff	$Q = C_f\,(CA)\,(I)$ Q = 1.0(0.47)(7.1) Q = 3.3 ft^3/sec	*Equation 1.3 of the Hydrology Technical Manual*; C_f = 1.0 (10yr) Col. 8 times Col. 11 multiplied by 1.0
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 1.18 or Figure 1.17 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 = 18.1ft^3/s	Equation 1.18 or Figure 1.17
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 1.17 or Figure 1.17
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 1.19a
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. 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 1.30
Col. 19 D/S Invert	= (365.5) - (361.0)(0.03) = 354.67 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 0	Upstream most invert
Col. 21 Slope	S = 0.03	select desired pipe slope

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 1.30
Col. 4 Inlet Area	A_i = 0.35 ac	Table 1.18
Col. 5 Total Area	A_t = 0.35 + 0.64 A_t = 0.99 ac	Col. 4 plus structure 42 total area, Table 1.18
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.73)(0.35) 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 1.18
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 1.19; system time equals 5 min
Col. 12 Runoff	$Q = (C_f)(CA)(I)$ Q = 1.0(0.72) (7.1) Q = 5.1 ft^3/sec	*Equation 1.3 of the Hydrology Technical Manual*; C_f =1.0(10-yr) Col. 8 times Col. 11 times C_f
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 1.18 or Figure 1.17 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 1.18 or Figure 1.17
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 1.18 or Figure 1.17
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 1.19a
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. 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)
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (0.5)(8.7)^2 / [(2)(32.2)]$ = 0.6 ft	Equation 1.36 with Table 1.15 $K_{ah} = 0.5$ for inlet - straight run
Col. 21 Slope	S = 0.03	select desired pipe slope

Structure 42 to 43

Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	L = 14.1 ft	Figure 1.30
Col. 4 Inlet Area	$A_i = 0.32$ ac	Table 1.18
Col. 5 Total Area	$A_t = 0.32 + 0.99$ $A_t = 1.31$ ac	Col. 4 plus previous Col. 5 total area, Table 1.18
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.73)(0.32) 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 43 total CA values
Col. 9 Inlet Time	$t_i = 2$ min	Table 1.18

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 1.19
Col. 12 Runoff	$Q = (C_f)(CA)(I)$ $Q = 1.0\ (0.95)\ (7.1)$ $Q = 6.75\ ft^3/sec$	Col. 8 times Col. 11
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 1.18 or Figure 1.17 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 1.18 or Figure 1.17
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 1.18 or Figure 1.17
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 1.19a
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. 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)
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (1.5)(2.6)^2 / [(2)(32.2)]$ = 0.16 ft	Equation 1.36 and Table 1.15; K_{ah} =1.5 for inlet - angled through 90 degrees
Col. 21 Slope	S = 0.001	Select desired pipe slope
Structure 43 to 44		
Col. 1 From	= 43	
Col. 2 To	= 44	
Col. 3 Run Length	L = 55.8 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.0 ac	Table 1.18

Col. 5 Total Area	A_t = 1.31 ac	Col. 4 plus previous Col. 5
Col. 6 "C"	C = n/a	Table 1.18
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 previous Col. 8
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 1.18
Col. 12 Runoff	$Q = C_f\ (CA)\ I$ $Q = 1.0\ (0.95)\ (7.1)$ $Q = 6.75\ ft^3/sec$	Col. 8 times Col. 11 times C_f
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 1.18 or Figure 1.17 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 1.18 or Figure 1.17
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 1.17 or Figure 1.17
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 1.19a
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
Col. 21 Slope	S = 0.01	Select desired pipe slope

(2) Energy Grade Line Evaluation Computations - English Units

The following computational procedure follows the steps outlined in *Section 1.2.11* above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of the computational process is provided in Figure 1.32 English and Figure 1.33 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 1.18
		$S_f = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^2$	
		$S_f = 0.00090$ ft/ft	
Step 7	Col. 2B	$H_f = S_f L$	Equation 1.27
		$H_f = (0.0009)(55.8)$	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)	(ft³/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 1.32 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 (1)	H_f (2)	h_b (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (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 1.33 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 TOC = 331.27 + 2.0 = 333.27	Check for full flow - close 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 1.32)
Step 21	Col. 16A	Surf. Elev. = 347.76 ft 347.76 > 333.09	From Figure 1.31. 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	 Col. 14A	HGL' = inv. + $(d_c+D)/2$ HGL' = 344.06 + (0.80 + 2.0)/2 HGL = 345.46 ft	Calculate new HGL - Use Case 2 d_c from Figure 1.19b
		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 1.32
	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 1.19a) d_c = 0.80 (HDS-5) Flow is subcritical
Step 7		$H_f = S_f L$ $H_f = (0.001)(14.1)$	Equation 1.27 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	$K_o = 0.1(b/D_o)(1-\sin\theta)+1.4(b/D_o)_{0.15}\sin(\theta)$	Equation 1.38
		b = 4.0 ft	Access hole diameter.
		D_o = 2.0 ft	Col. 2A - outlet pipe diam
		θ = 90°	Flow deflection angle
		$K_o = 0.1(4.0/2.0)(1 - \sin 90)+$ $1.4(4.0/2.0)^{0.15}\sin 90$ K_o = 1.55	
Step 11	Col. 10B	$C_D = (D_o/D_i)^3$	Equation 1.39; pipe diameter
		d_{aho} = 1.40	Column 8B
		d_{aho}/D_o = (1.40/2.0)	
		d_{aho}/D_o = 0.70 < 3.2	therefore
		C_D = 1.0	

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 1.40; 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 1.41; 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 1.42; plunging flow No plunging flow
Step 15	Col. 14B	$C_B = 1.0$	Benching Correction, flat floor (Table 1.17)
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 1.37
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 1.31)
Step 21	Col 16A	Surf. Elev. = 349.31 ft 349.31 > 345.96	From Figure 1.30 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	 Col. 6aA	$Q/Q_f = 5.1/18.1 = 0.28$ $d/d_f = 0.37$ d = (0.37) (1.5) d = 0.56 ft	 Figure 1.19a
	  Col. 5A	$V/V_f = 0.84$ V = (0.84)(10.3) V = 8.65 fps	Figure 1.19a

		$V^2/2g = (8.65)^2/(2)(32.2)$	Velocity head
	Col. 7A	$V^2/2g = 1.16$ ft	
Step 5B	Col. 6bA	$d_c = 0.85$ ft	Figure 1.19b
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 1.19a
		d = (0.29)(1.5)	
	Col. 6aA	d = 0.43 ft	
	Col. 6bA	$d_c = 0.7$ ft	Figure 1.19b
Step 5H		$V/V_f = 0.73$	Figure 1.19a
		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 1.31) for Str. 41
		U/S TOC = 356.17 ft	
Step 21	Col. 16A	Surf. Elev. = 360.0 ft	From Figure 1.30.
		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 1.31, 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 1.31) for Str. 40
Step 21	Col. 16A	Surf. Elev. = 370.0 ft 370.0 ft > 366.50 ft	From Figure 1.30 Surface Elev. > HGL, OK

See Figures 1.32 and 1.33 for the tabulation of results. The final HGL values are indicated in Figure 1.30.

1.3 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.

American Society of Civil Engineers, 1993, Design and Construction of Urban Stormwater Management Systems, Manual and Report No. 77.

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.

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

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, "Stormwater 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.

Larry W. Mays, Editor, 2001, Stormwater Collection Systems Design Handbook, McGraw-Hill Companies, Inc., New York, NY

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.

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

Texas Department of Transportation, November 2002, Hydraulic Design Manual, Austin, Texas, page 5-17.

The Dewberry Companies, 2002, 2nd Edition, Land Development Handbook, McGraw-Hill Companies, Inc., New York, NY.

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.

2.0 Storage Design

2.1 General Storage Concepts

2.1.1 Introduction

This section provides general guidance on stormwater runoff storage for meeting stormwater management control objectives (i.e., water quality protection, downstream streambank protection, and flood control).

Storage of stormwater runoff within a stormwater 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 stormwater controls and/or nonstructural features and landscaped areas. Figure 2.1 illustrates various storage facilities that can be considered for a development site.

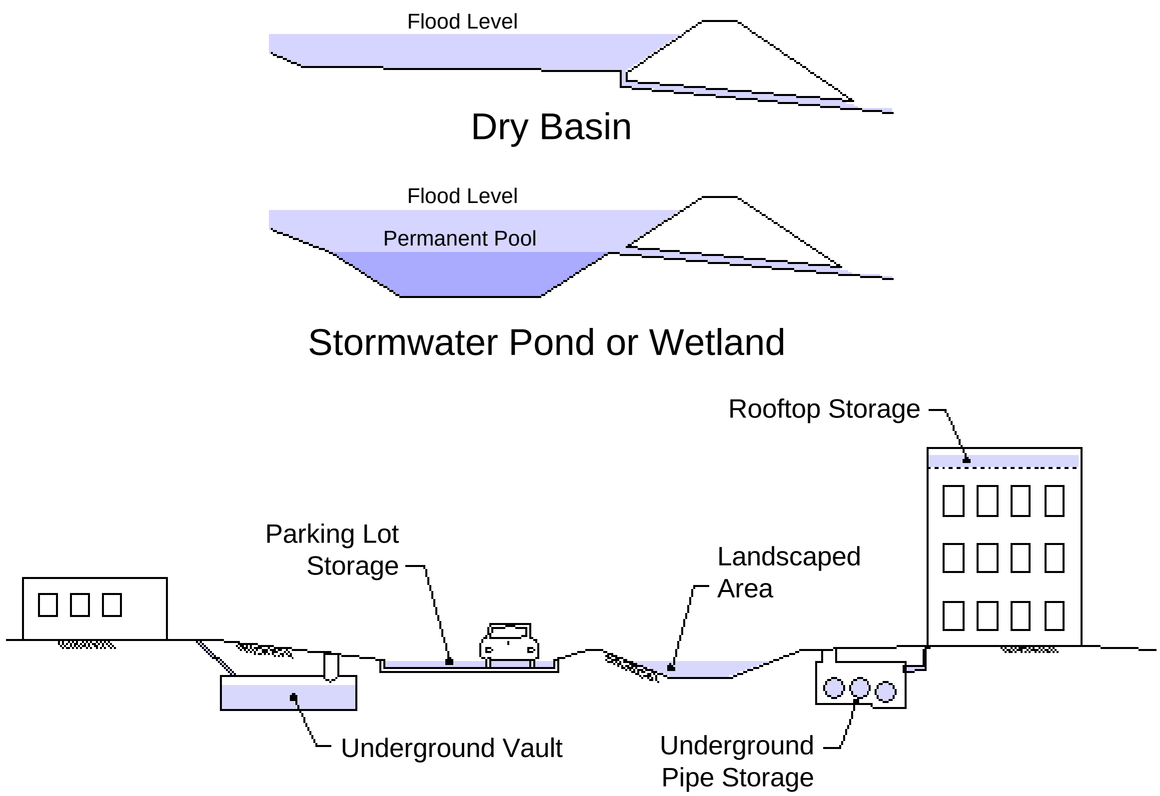


Figure 2.1 Examples of Typical Stormwater Storage Facilities

2.1.2 Storage Classification

Stormwater storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

Stormwater *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 stormwater 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 stormwater runoff from multiple projects and/or properties. A discussion of regional stormwater controls is found in *Section 1.0 of the Site Development Controls Technical Manual*.

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 2.2 illustrates on-line versus off-line storage.

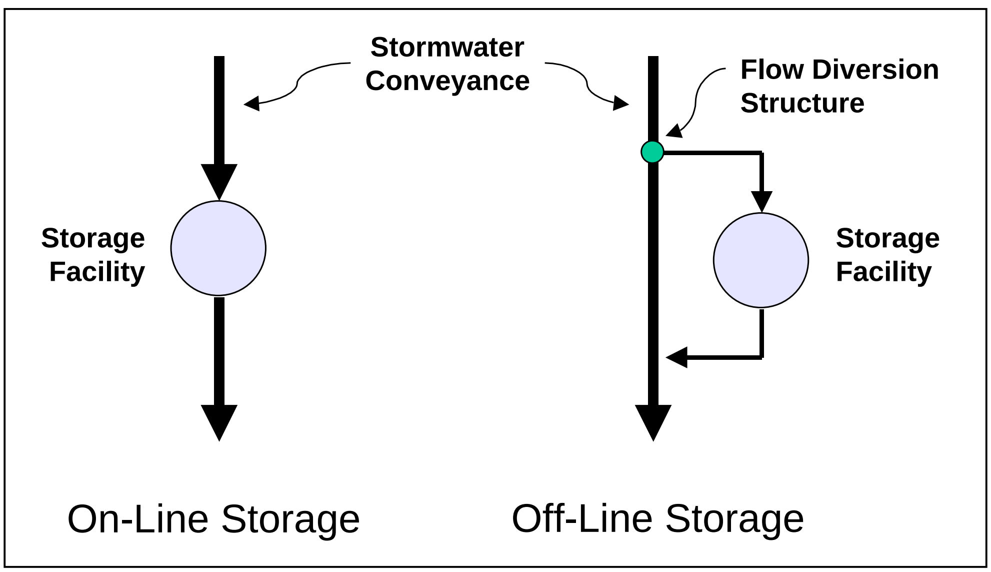


Figure 2.2 On-Line versus Off-Line Storage

2.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 2.3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

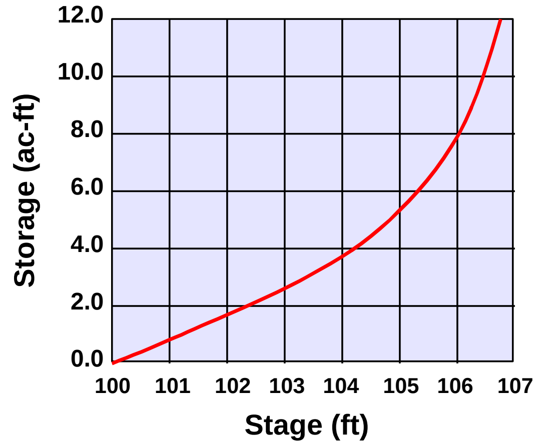


Figure 2.3 Stage-Storage Curve

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 2.4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \tag{2.1}$$

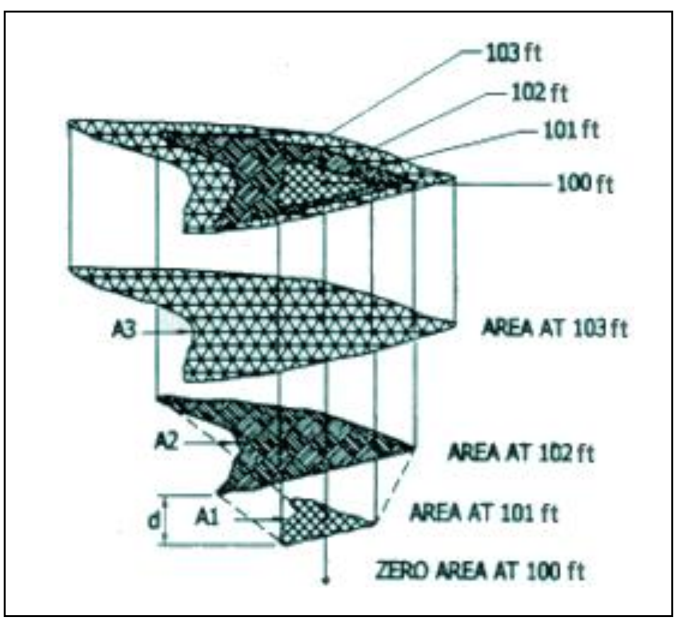


Figure 2.4 Double-End Area Method

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:

$$\mathbf{V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3} \quad \textbf{(2.2)}$$

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:

$$\mathbf{V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3} \quad \textbf{(2.3)}$$

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:

$$\mathbf{V = 1.047\ D\ (R_1^2 + R_2^2 + R_1R_2)} \quad \textbf{(2.4)}$$

$$\mathbf{V = 1.047\ D\ (3\ R_1^2 + 3ZDR_1 + Z_2D^2)} \quad \textbf{(2.5)}$$

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

2.1.4 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 2.5). 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 2.2*.

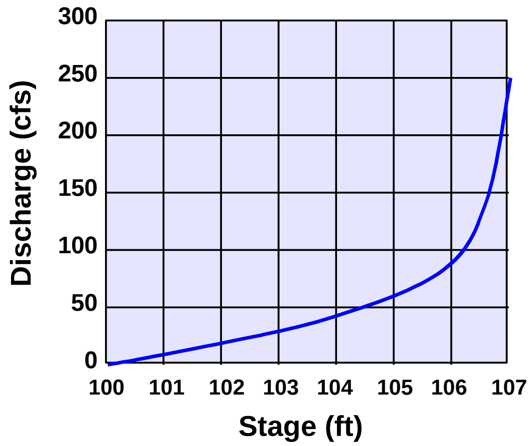


Figure 2.5 Stage-Discharge Curve

2.1.5 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 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 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	-

2.1.6 General Storage Design Procedures

Introduction

This section discusses the general design procedures for designing storage to provide standard detention of stormwater 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 stormwater 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 *Section 2.0 of the Hydrology Technical Manual*).

In multi-purpose multi-stage facilities such as stormwater ponds, the design of storage must be integrated with the overall design for water quality protection objectives. See *Section 1.0 of the Site Development Controls Technical Manual* for further guidance and criteria for the design of structural stormwater controls.

Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms (this can be generated using a unit hydrograph method or the Modified Rational Method, see the *Hydrology Technical Manual* for more details)
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

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 flood mitigation (Q_{p100}) design storms using the hydrologic methods outlined in *Section 1.0 of the Hydrology Technical Manual*. Both existing- and post-development hydrographs are required for both the "Conveyance" and flood mitigation design storms.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see *Section 2.1.7*).

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 flood mitigation 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 flood mitigation storm. If not then consider emergency overflow from runoff due to the flood mitigation (or larger) design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflows for the “Conveyance” and flood mitigation 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 *Section 2.0 of the Hydrology Technical Manual*).

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.

2.1.7 Preliminary Detention Calculations

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.

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 2.6.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$\mathbf{V_S = 0.5T_i\,(Q_i - Q_O)} \qquad \textbf{(2.6)}$$

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)

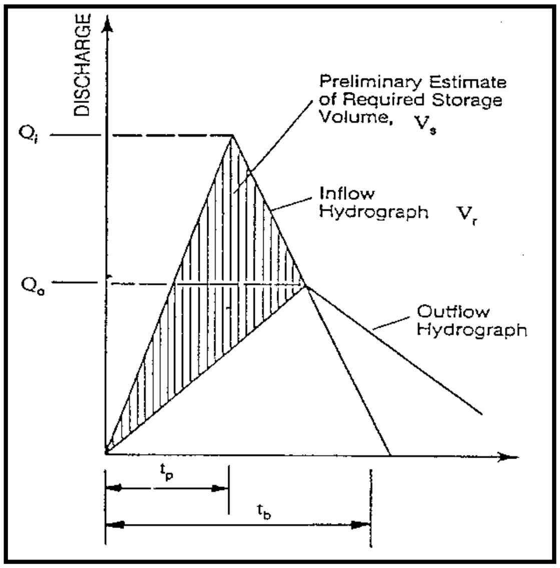


Figure 2.6 Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

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:

$$V_s/V_r = \frac{1.291(1 - Q_o/Q_i)^{0.753}}{(t_b/t_p)^{0.411}} \qquad (2.7)$$

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.

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):

$$Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \qquad \textbf{(2.8)}$$

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.

2.2 Outlet Structures

2.2.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.2 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 2.2 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

2.2.2 Primary Outlets

Introduction

Primary outlets provide the critical function of the regulation of flow for structural stormwater 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 stormwater 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 stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

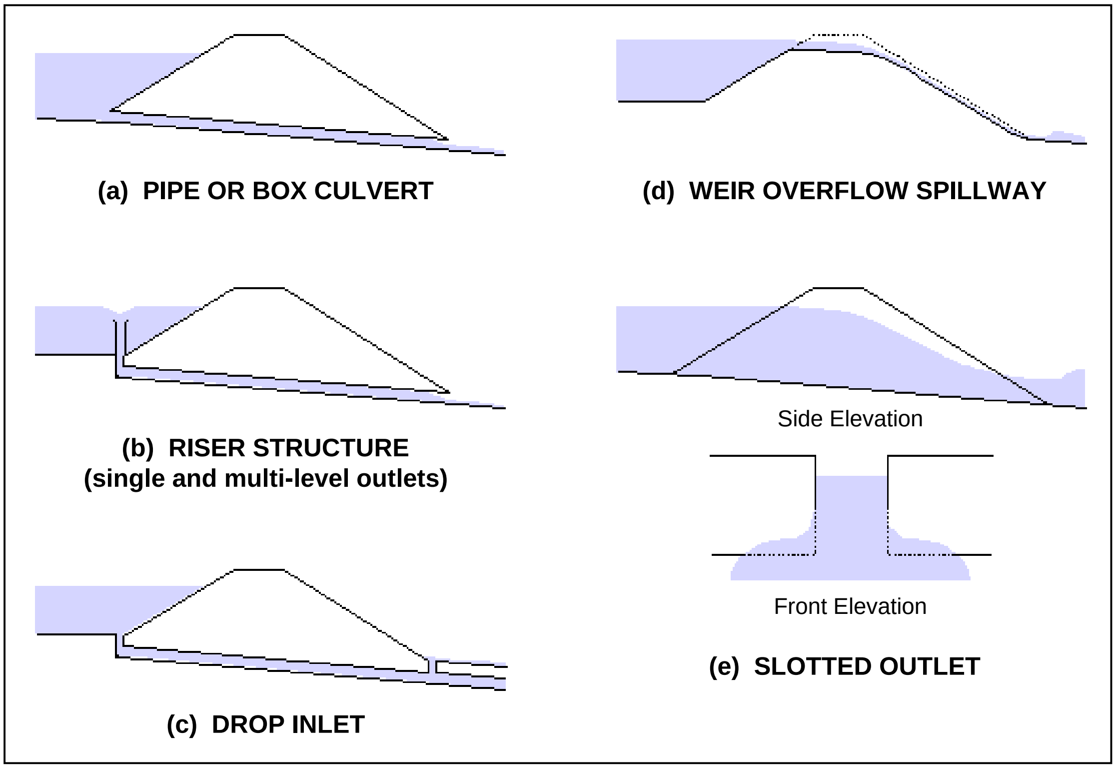


Figure 2.7 Typical Primary Outlets

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 stormwater 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 stormwater 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.

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 2.8(a), the orifice discharge can be determined using the standard orifice equation below.

$$\mathbf{Q = CA\,(2gH)^{0.5}} \qquad \mathbf{(2.9)}$$

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)

H = effective head on the orifice, from the center of orifice to the water surface (ft)

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 2.8(b).

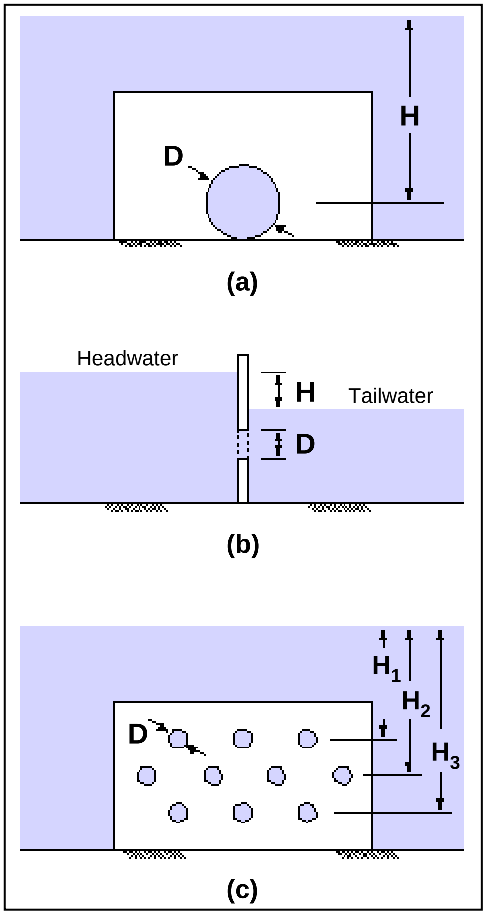


Figure 2.8 Orifice Definitions

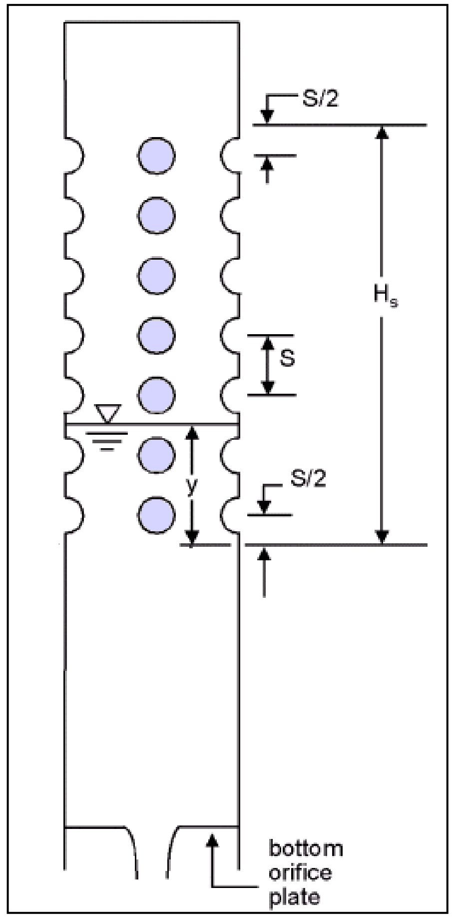


Figure 2.9 Perforated Riser

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:

$$Q = 0.6A\,(2gH)^{0.5} = 3.78D^2H^{0.5} \qquad (2.10)$$

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 2.8(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 2.3 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 2.3 Circular Perforation Sizing

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in²)		
		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 2.10 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.

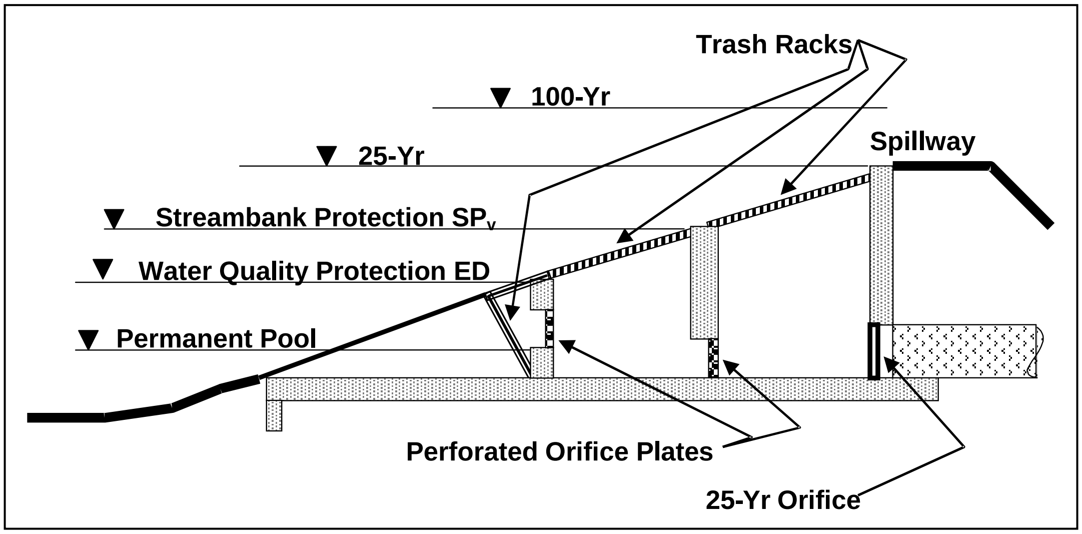


Figure 2.10 Schematic of Orifice Plate Outlet Structure

Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 2.9. 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 2.9, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p[(2A_p)/(3H_s)]*(\sqrt{2g})*H^{3/2} \qquad (2.11)$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft²)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

Pipes and Culverts

Discharge pipes are often used as outlet structures for stormwater 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. 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 3.3*, or by using Equation 2.12 (NRCS, 1984).

The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$\mathbf{Q = a[(2gH) / (1 + k_m + k_pL)]0.5} \qquad \textbf{(2.12)}$$

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)

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 2.11(a). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)] LH^{1.5}} \qquad \textbf{(2.13)}$$

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)

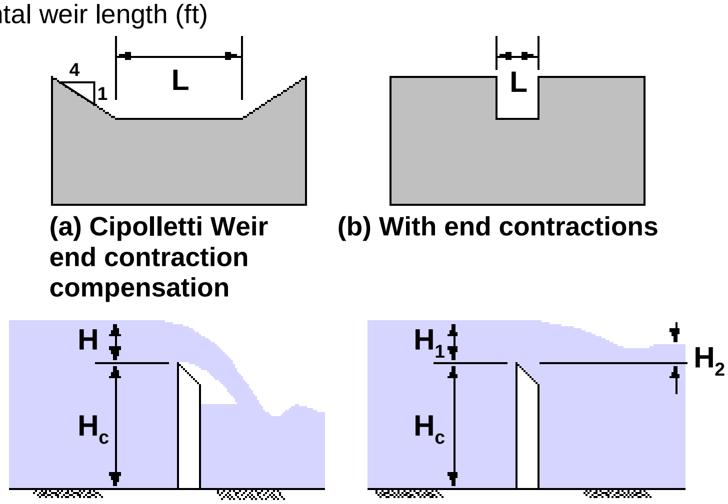


Figure 2.11 Sharp-Crested Weir

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 2.11(b). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)]\ (L - 0.2H)\ H^{1.5}} \qquad \textbf{(2.14)}$$

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):

$$\mathbf{Q_S = Q_f\ (1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \textbf{(2.15)}$$

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)

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):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(2.16)}$$

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 2.4.

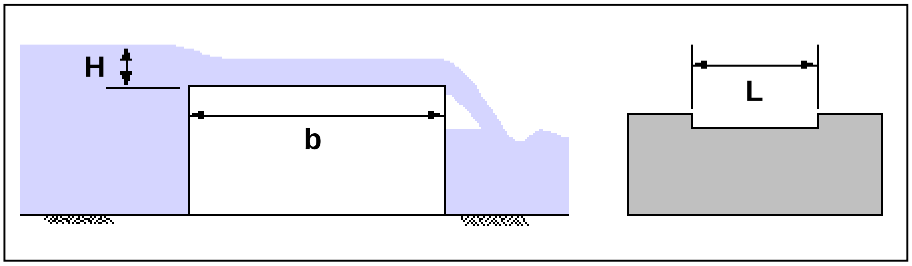


Figure 2.12 Broad-Crested Weir

Table 2.4 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)

V-Notch Weirs

The discharge through a V-notch weir (Figure 2.13) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)\, H^{2.5} \qquad (2.17)$$

where:

Q = discharge (cfs)

θ = angle of V-notch (degrees)

H = head on apex of notch (ft)

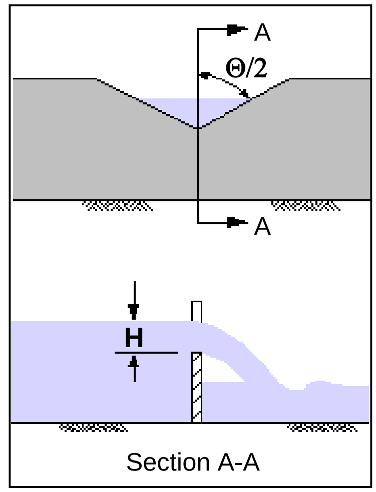


Figure 2.13 V-Notch Weir

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 2.14. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\, a^{0.5}\, b\, (H - a/3) \qquad (2.18)$$

$$x/b = 1 - (1/3.17)\left(\arctan\,(y/a)^{0.5}\right) \qquad (2.19)$$

where:

Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 2.14

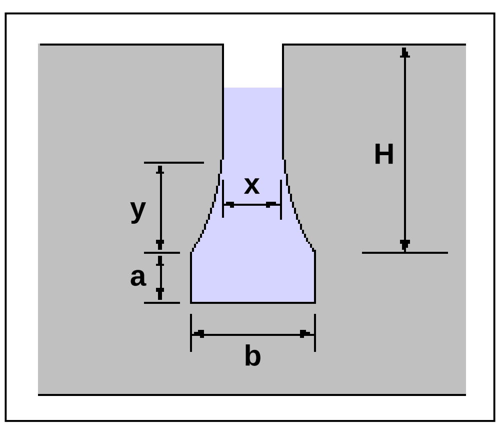


Figure 2.14 Proportional Weir Dimensions

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 2.15 shows an example of a riser designed for a wet extended detention pond. The orifice plate outlet structure in Figure 2.9 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 2.16) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

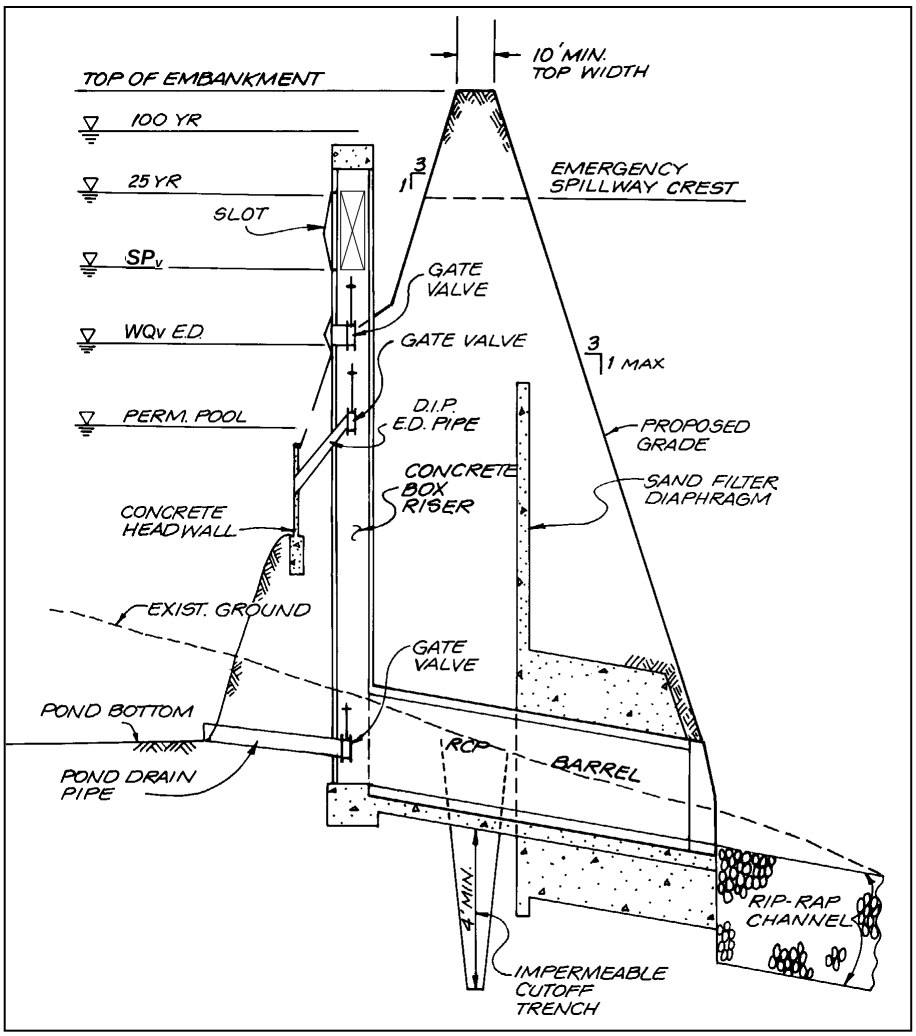


Figure 2.15 Schematic of Combination Outlet Structure

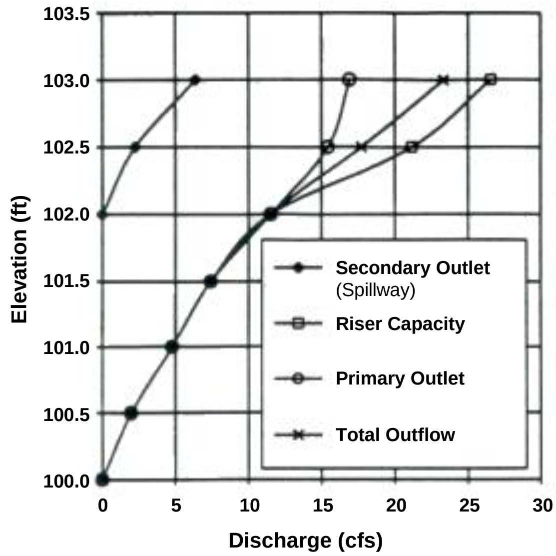


Figure 2.16 Composite Stage-Discharge Curve

2.2.3 Extended Detention (Water Quality and Streambank Protection) Outlet Design

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.

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 2.9 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.

Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example 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 2.9 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.

2.2.4 Multi-Stage Outlet Design

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 stormwater ponds, stormwater 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 2.9 and 2.15 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 2.16)

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 Stormwater Control Volumes. Using the procedures from *Section 1.0 of the Hydrology and Water Quality Technical Manuals*, 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 *Section 2.2.3*. 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 *Section 2.2.5*).

Step 4 Design Streambank Protection Outlet. Design the streambank protection extended detention outlet (SP_v-ED) using either method from *Section 2.2.3*. 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 2.17, 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 2.17 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 2.18 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 stormwater impoundment structures have a vegetated emergency spillway (see *Section 2.2.7*). 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 flood mitigation 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 *Section 4.0*, 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.

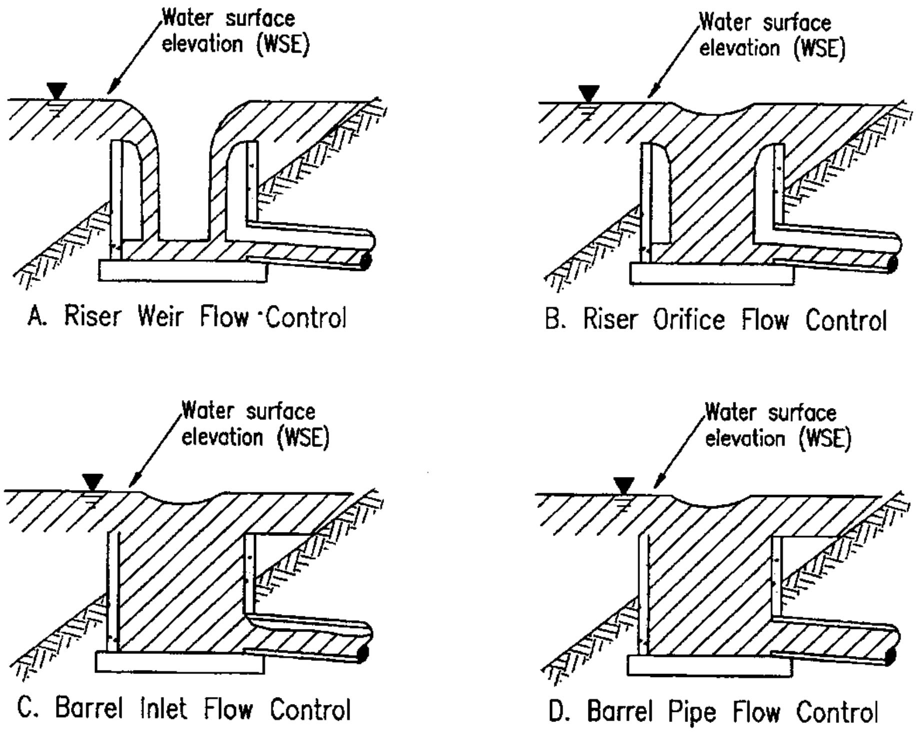


Figure 2.17 Riser Flow Diagrams
(Source: VDCR, 1999)

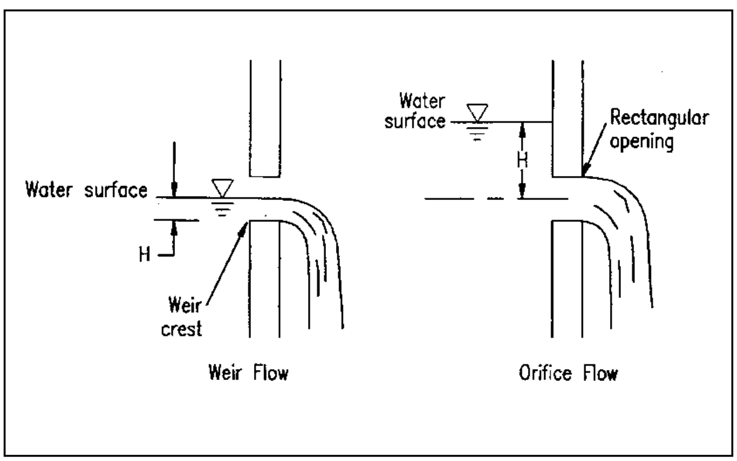


Figure 2.18 Weir and Orifice Flow
(Source: VDCR, 1999)

2.2.5 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 stormwater pond or wetland with a permanent pool (see Figure 2.19). 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 stormwater pond or wetland with a permanent pool is shown in Figures 2.20 and 2.21.

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 2.22).

Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

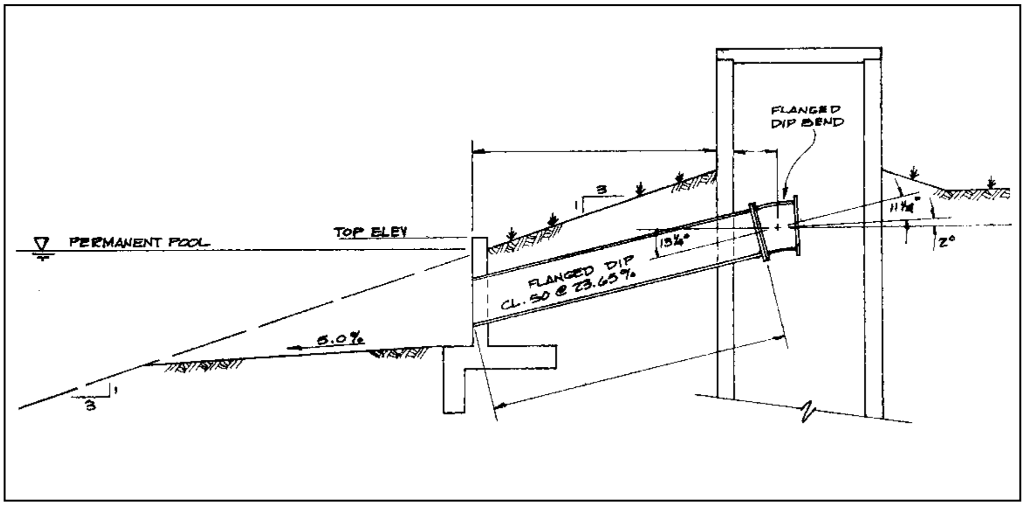


Figure 2.19 Reverse Slope Pipe Outlet

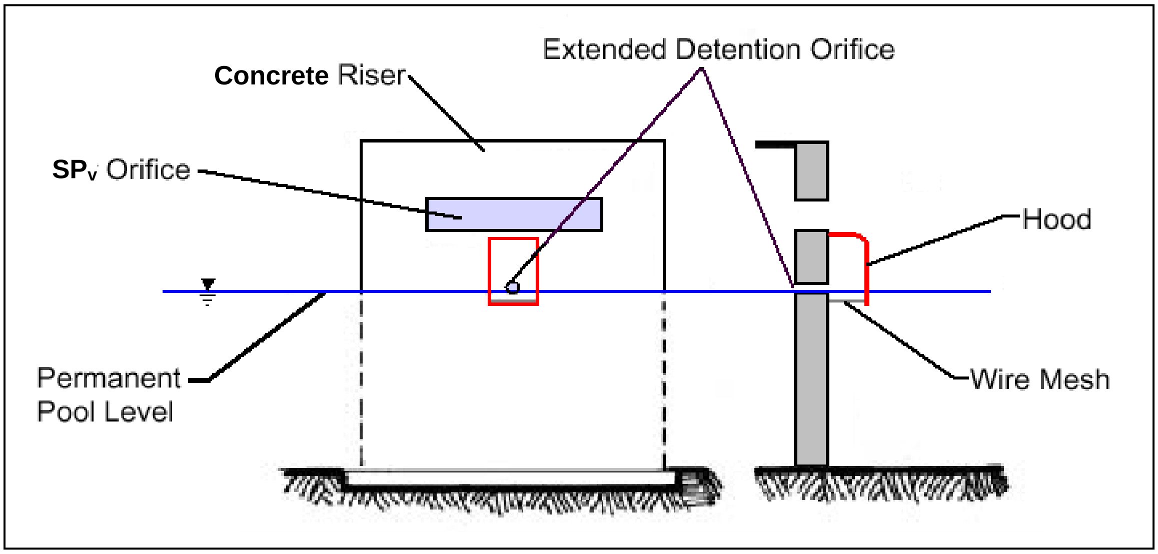


Figure 2.20 Hooded Outlet

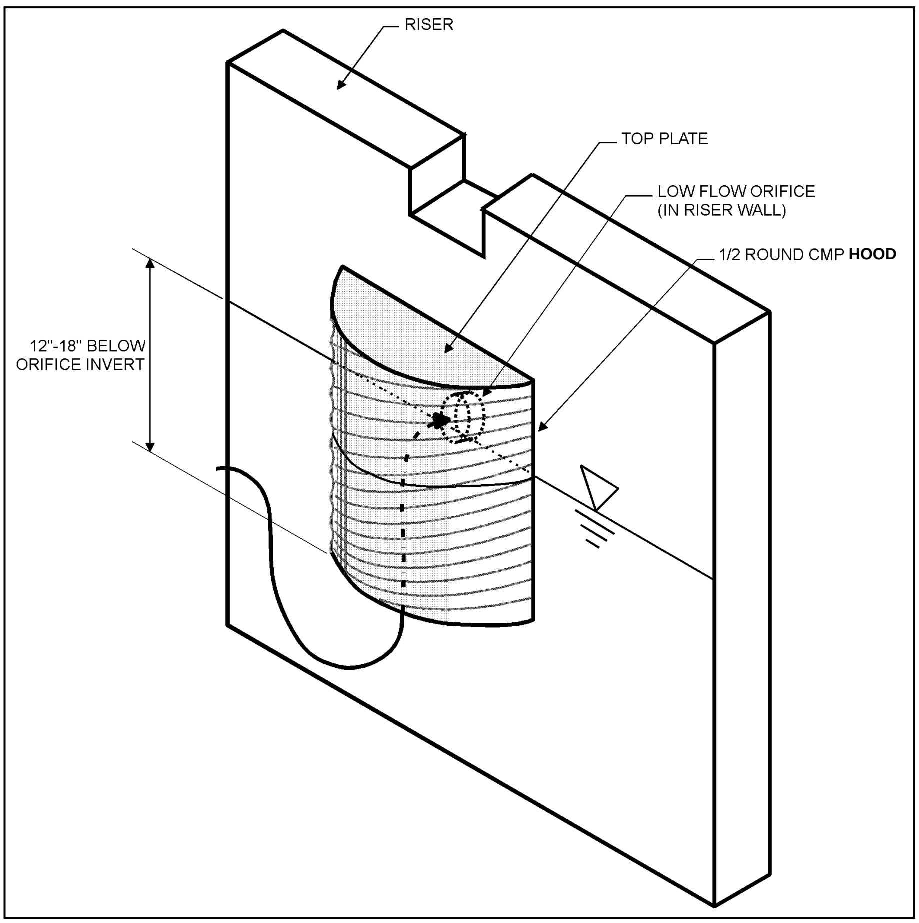


Figure 2.21 Half-Round CMP Orifice Hood

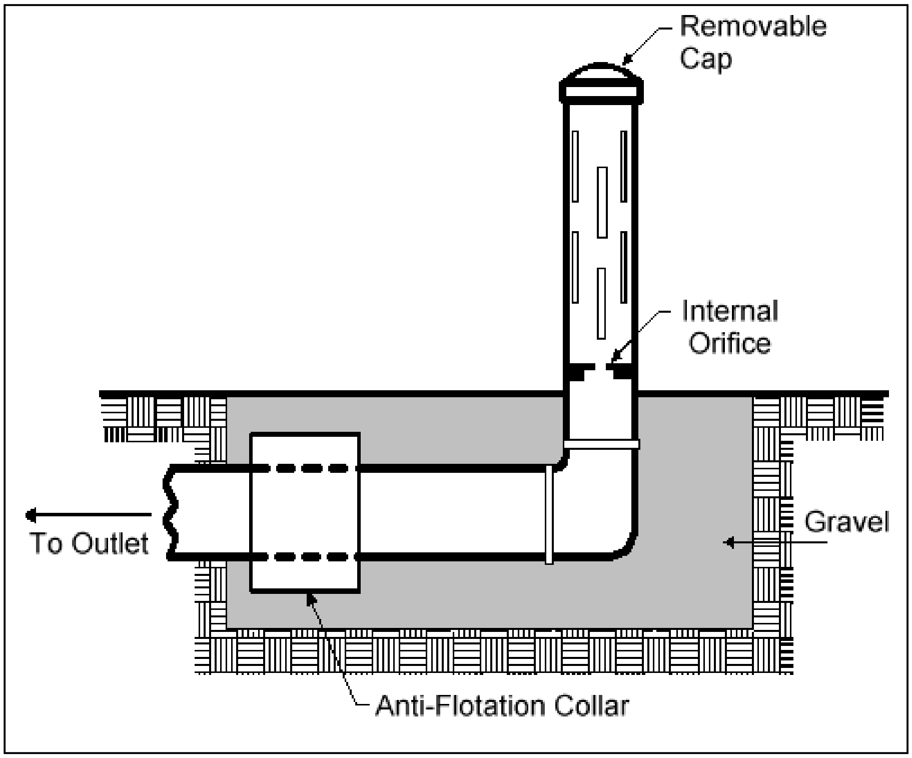


Figure 2.22 Internal Control for Orifice Protection

2.2.6 Trash Racks and Safety Grates

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 2.23. 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.

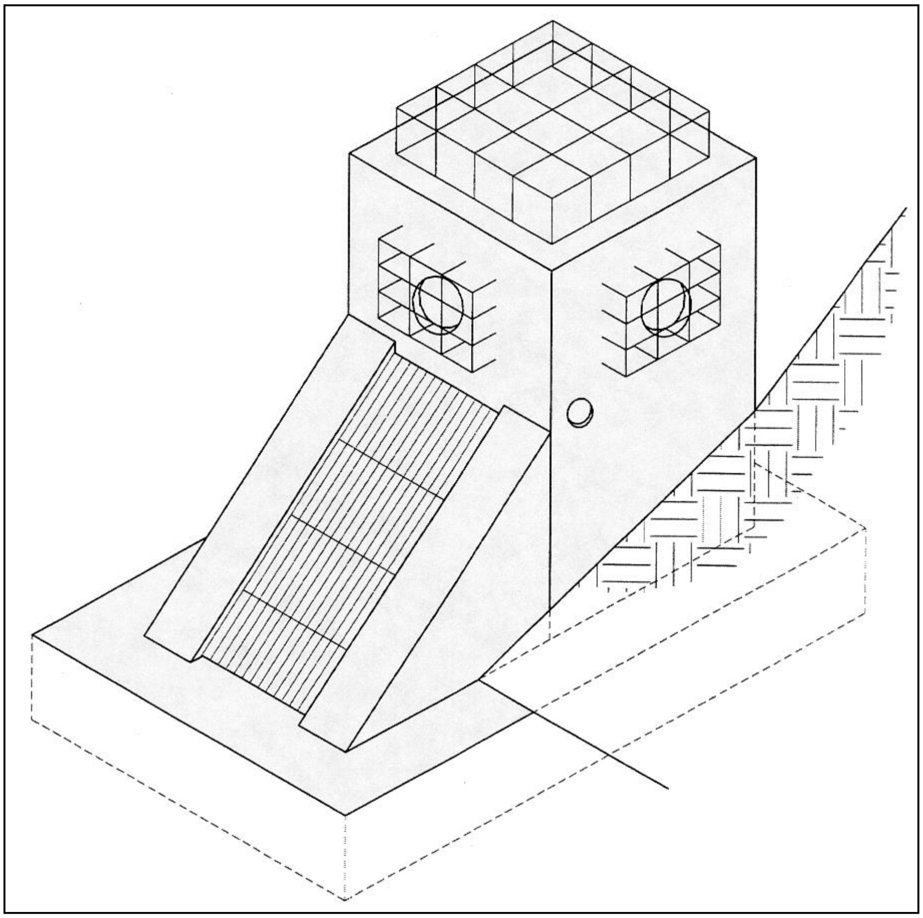


Figure 2.23 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

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 2.24 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.

$$\mathbf{H_g = K_{g1}\,(w/x)^{4/3}\,(V_u^2/2g)\,\sin\theta_g} \qquad \mathbf{(2.20)}$$

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.

$$\mathbf{H_g = \frac{K_{g2}V_u^2}{2g}} \qquad \mathbf{(2.21)}$$

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.

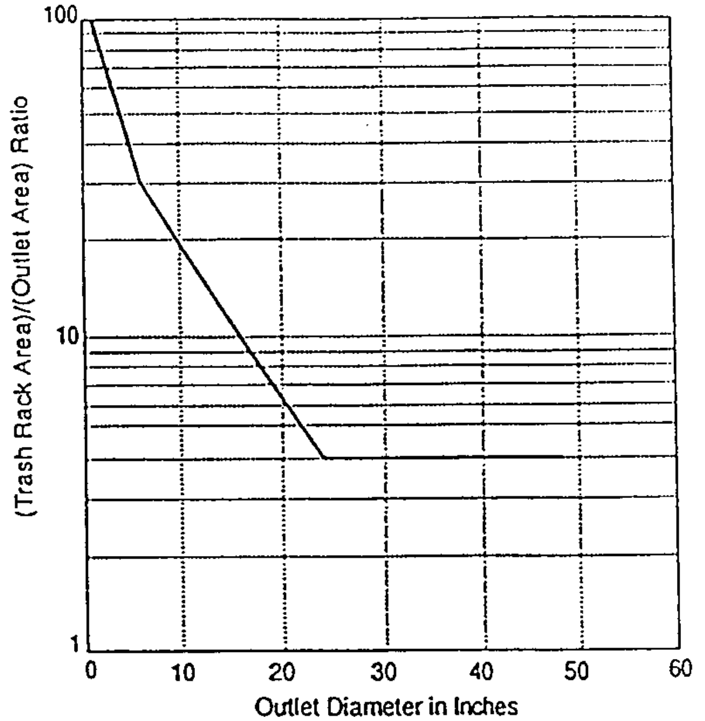


Figure 2.24 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

2.2.7 Secondary Outlets

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 2.25 shows an example of an emergency spillway.

In many cases, on-site stormwater 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.

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 2.25). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the flood mitigation storm 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 flood mitigation storm 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 3.2*, 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.

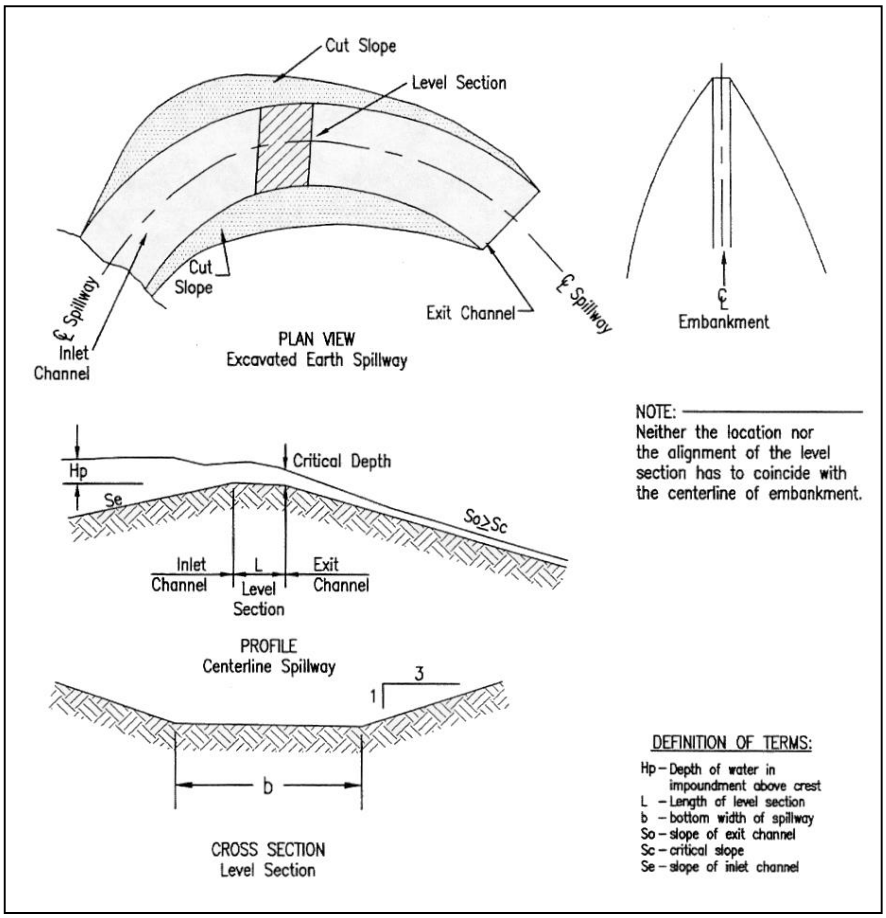


Figure 2.25 Emergency Spillway
(Source: VDCR, 1999)

2.3 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 Stormwater 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.

3.0 Open Channels, Culverts and Bridges

3.1 Open Channels, Culverts and Bridges Overview

3.1.1 Key Issues in Stormwater System Design

Introduction

The traditional design of stormwater systems has been to collect and convey stormwater 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:

- Stormwater 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 stormwater controls to mitigate the major stormwater impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in stormwater system design.

General Design Considerations

- Stormwater 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 stormwater 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 stormwater (minor) systems.
- It is important to ensure that the combined on-site flood control system and major stormwater 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 stormwater systems and/or major stormwater structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of stormwater systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major stormwater system design capacity.

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 3.1 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.

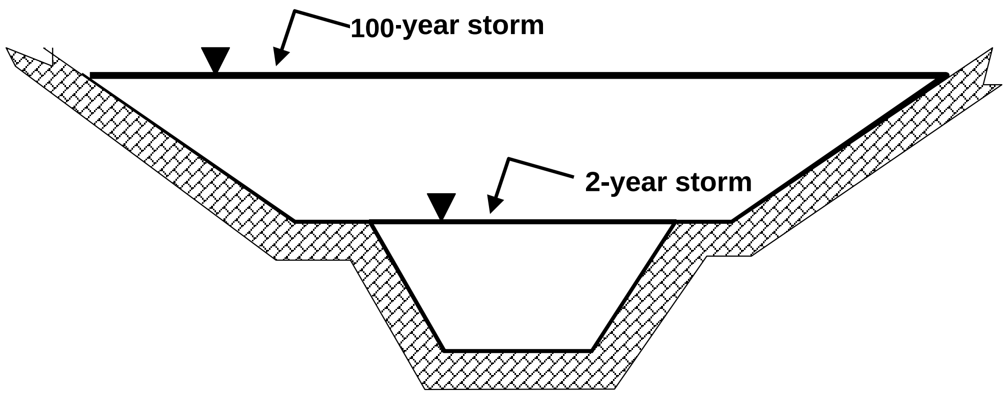


Figure 3.1 Compound Channel

- 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.

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.

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.

Storage Design

- Stormwater storage within a stormwater 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 stormwater controls and/or nonstructural features.
- Stormwater storage can be provided by detention, extended detention, or retention.

- Storage facilities may be provided on-site, or as regional facilities designed to manage stormwater runoff from multiple projects.

Outlet Structures

- Outlet structures provide the critical function of the regulation of flow for structural stormwater 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.

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.

3.2 Open Channel Design

3.2.1 Overview

Introduction

Open channel systems and their design are an integral part of stormwater 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.

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 3.6.3 of the Criteria Manual* and the *Site Development Controls Technical Manual* 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.

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.

3.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.1 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 3.1 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	-

3.2.3 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 3.2. Recommended values for vegetative linings should be determined using Figure 3.2, 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 3.6). Figure 3.2 is used iteratively as described in *Section 3.2.5*. Recommended Manning's values for natural channels that are either excavated or dredged, and natural are given in Table 3.2. 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. When designing open channels, the usual choice of Manning's roughness coefficients may be found in Table 3.5. The local jurisdiction may choose to vary from these values.

Table 3.2 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

Table 3.2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
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 vegetation on banks. Values of "n" for larger streams of mostly regular sections, with no boulders or brush	Range from 0.028 to 0.060	3 to 6
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 3.3 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 >10	5 4
Grass mixture	0-5[1] 5-10	4 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 3.4 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.

Table 3.5 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

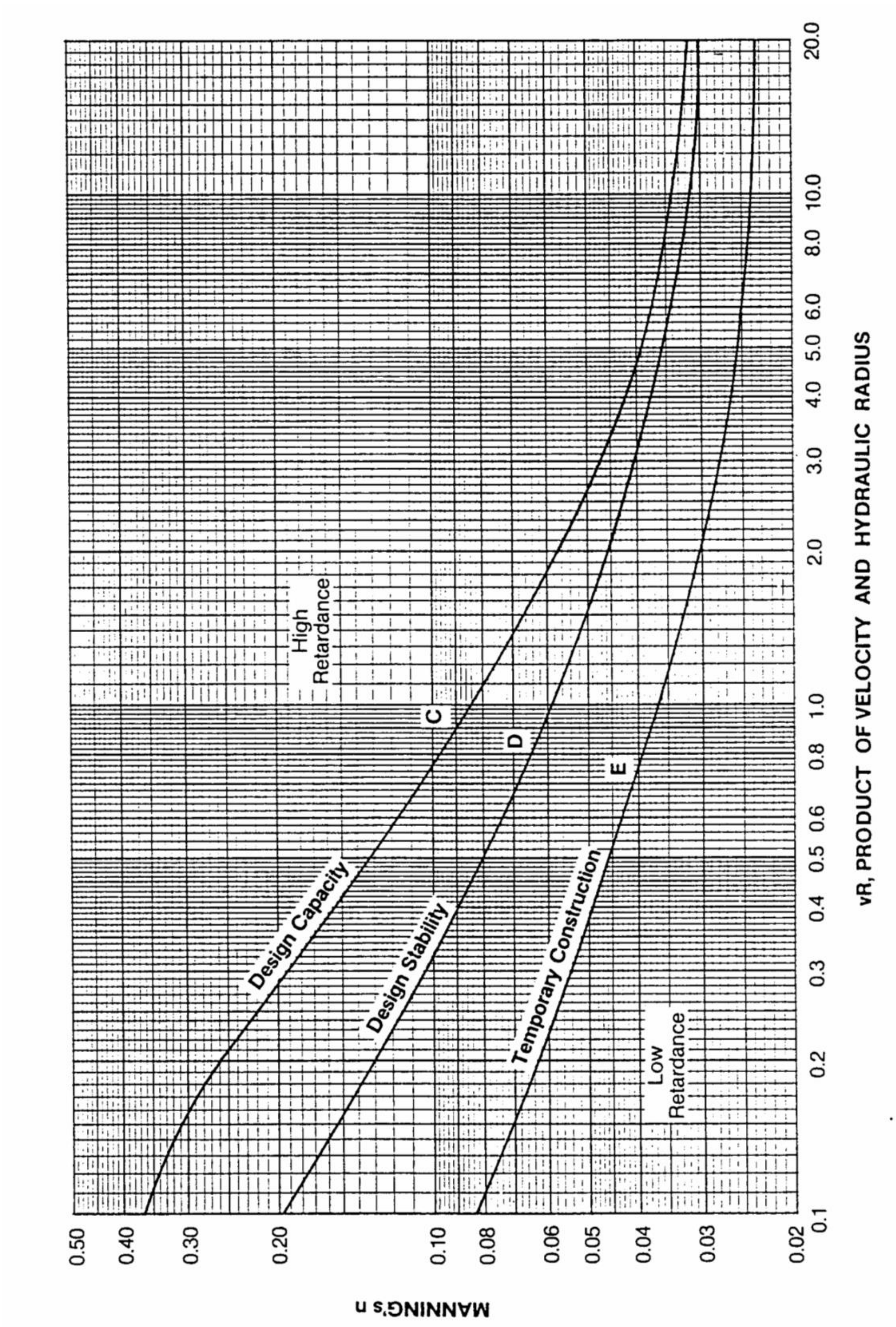


Figure 3.2 Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 3.6 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

3.2.4 Uniform Flow Calculations

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 *Section 3.2.11*.

Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$\mathbf{v = (1.49/n)\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(3.1)}$$

$$\mathbf{Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(3.2)}$$

$$\mathbf{S = [Q_n/(1.49\, A\, R^{2/3})]^2} \qquad \mathbf{(3.3)}$$

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.

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.

Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 3.2. The slope can be calculated using Equation 3.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 3.3 and 3.4. Figure 3.3 provides a general solution for the velocity form of Manning's Equation, while Figure 3.4 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 3.3:

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.

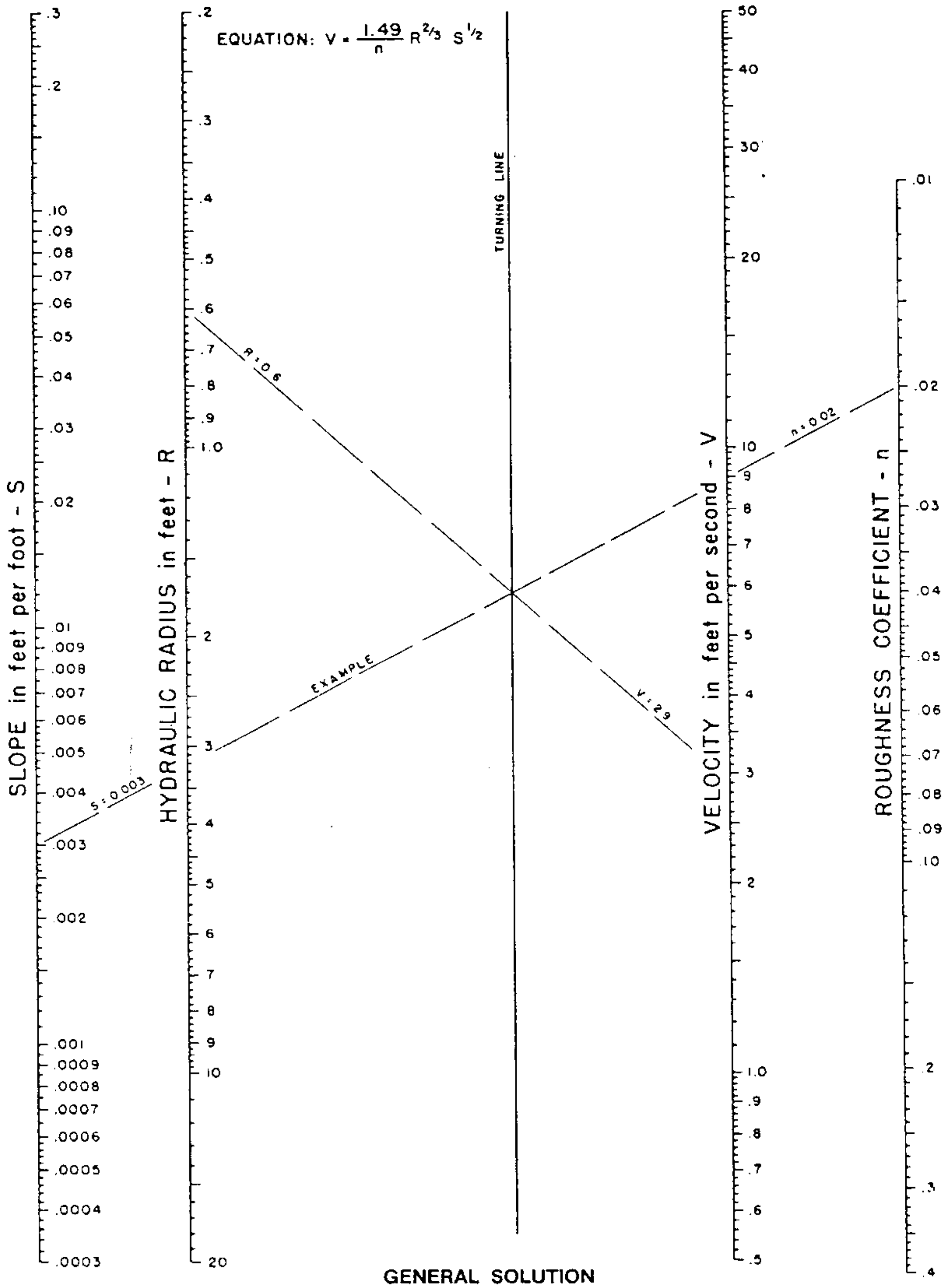
Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 3.4 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.

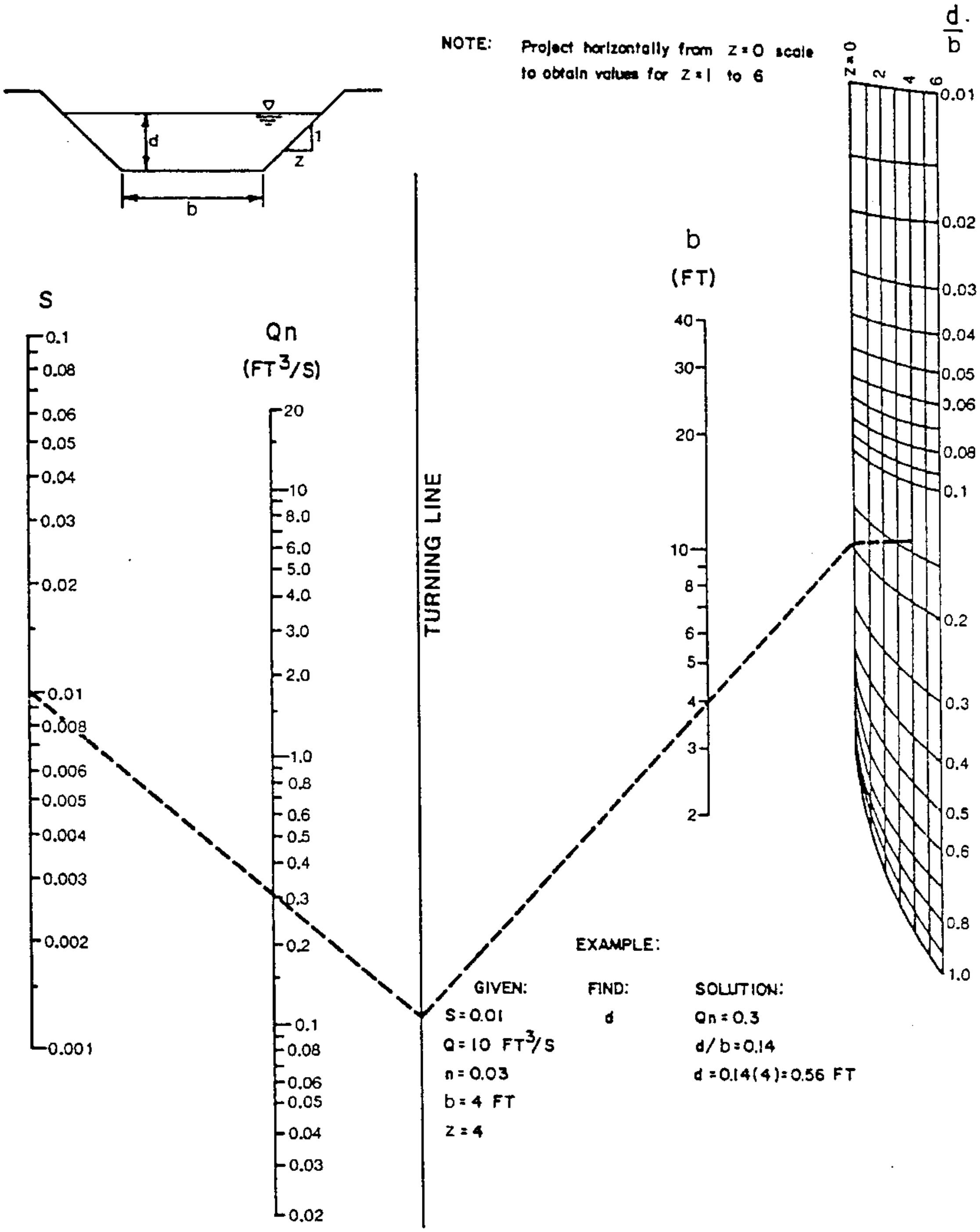
Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

- 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.
- Given d, find Q
  a. 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.
  b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.
  c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.
  d. 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 3.3 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3.4 Solution of Manning's Equation for Trapezoidal Channels

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:

$$\mathbf{AR^{2/3} = (Qn)/(1.49\ S^{1/2})} \qquad \textbf{(3.4)}$$

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 3.4 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 3.5 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:

$$\mathbf{K_T = (Qn)/(b^{8/3}S^{1/2})} \qquad \textbf{(3.5)}$$

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 3.5 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 3.11

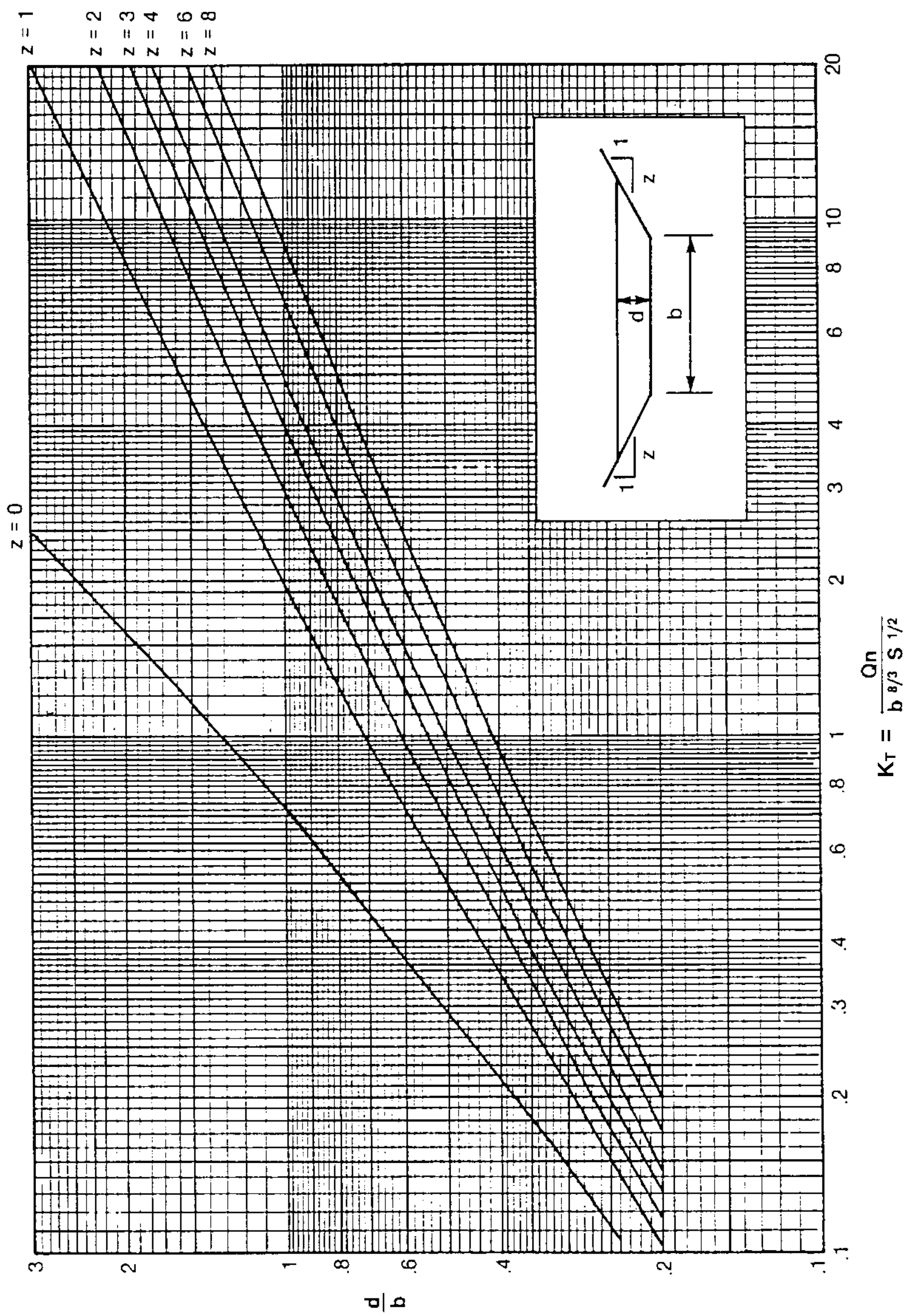


Figure 3.5 Trapezoidal Channel Capacity Chart
(Source: Nashville Stormwater Management Manual, 1988)

3.2.5 Critical Flow Calculations

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:

$$Q^2/g = A^3/T \qquad (3.6)$$

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 3.6.

Semi-Empirical Equations

Semi-empirical equations (as presented in Table 3.7) or section factors (as presented in Figure 3.6) 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:

$$Z = Q/(g^{0.5}) \qquad (3.7)$$

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:

- A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
- If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
- If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
- 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:

$$Fr = v/(gA/T)^{0.5} \tag{3.8}$$

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 3.7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

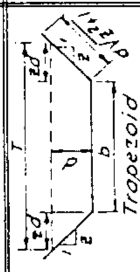
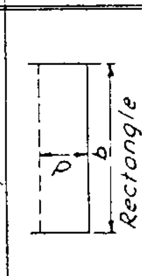
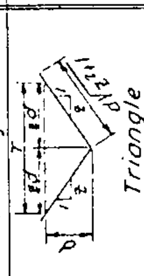
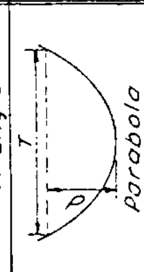
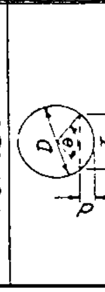
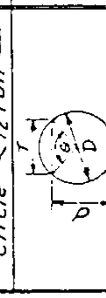
Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel 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 3.6 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 - < ½ 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 - > ½ 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 3.6 Open Channel Geometric Relationships for Various Cross Sections

3.2.6 Vegetative Design

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 3.6. 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 3.6. 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).

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 3.3 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 3.2. Use retardance Class D for permanent vegetation and E for temporary construction.

Step 4 Calculate the hydraulic radius using the equation:

$$\mathbf{R = (vR)/v_m} \qquad \textbf{(3.9)}$$

where:

R = hydraulic radius of flow (ft)

vR = value obtained from Figure 3.2 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:

$$\mathbf{vR = (1.49\ R^{5/3}\ S^{1/2})/n} \qquad \textbf{(3.10)}$$

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 3.2 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 3.4 or 3.5, as described in *Section 3.2.4*. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in *Section 3.2.4*.

Step 8 If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 3.7. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

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 3.6 for equations).

Step 2 Divide the design flow rate, obtained using appropriate procedures from the *Hydrology Technical Manual*, 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 3.2 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 3.1) or Figure 3.3 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.

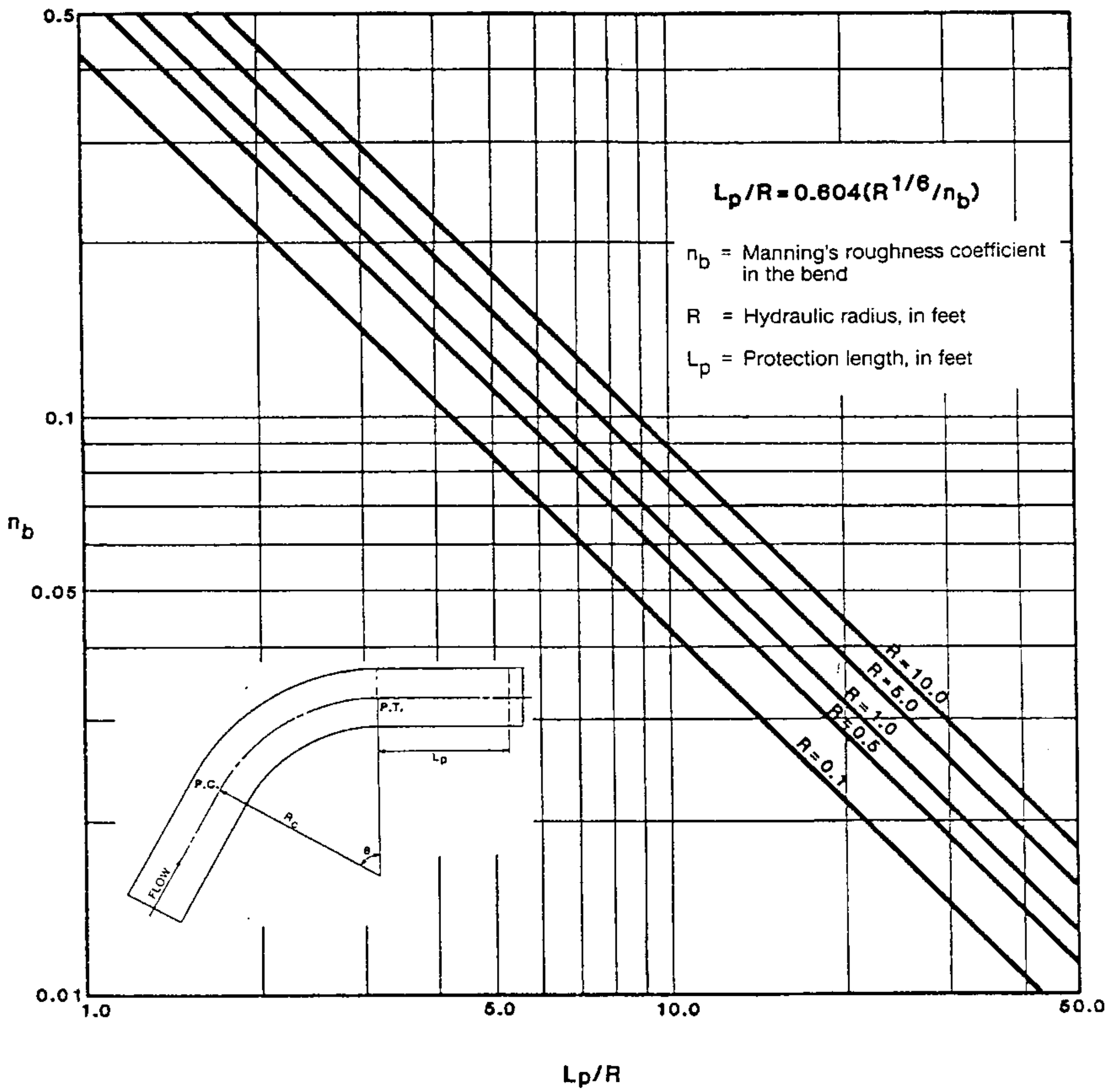
Step 8 If bends are considered, calculate super-elevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2T)/(gR_c) \qquad (3.11)$$

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 3.7 Protection Length, Lp, Downstream of Channel Bend

3.2.7 Stone Riprap Design

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.

The following methods give design guidance for designing stone riprap for open channels. Design guidance for designing stone riprap for culvert outfall protection is also provided in this section. *Section 4.0* gives additional guidance on the design of riprap aprons for erosion protection at outfalls, and the design of riprap basins for energy dissipation.

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:

$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \quad \mathbf{(3.12)}$$

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 3.8 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 3.9.

Step 4 Determine the required minimum d_{30} value from Figure 3.10, or from the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \quad \textbf{(3.13)}$$

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 3.8), dimensionless

v = mean velocity above the stone (ft/s)

g = acceleration of gravity (32.2 ft/sec)

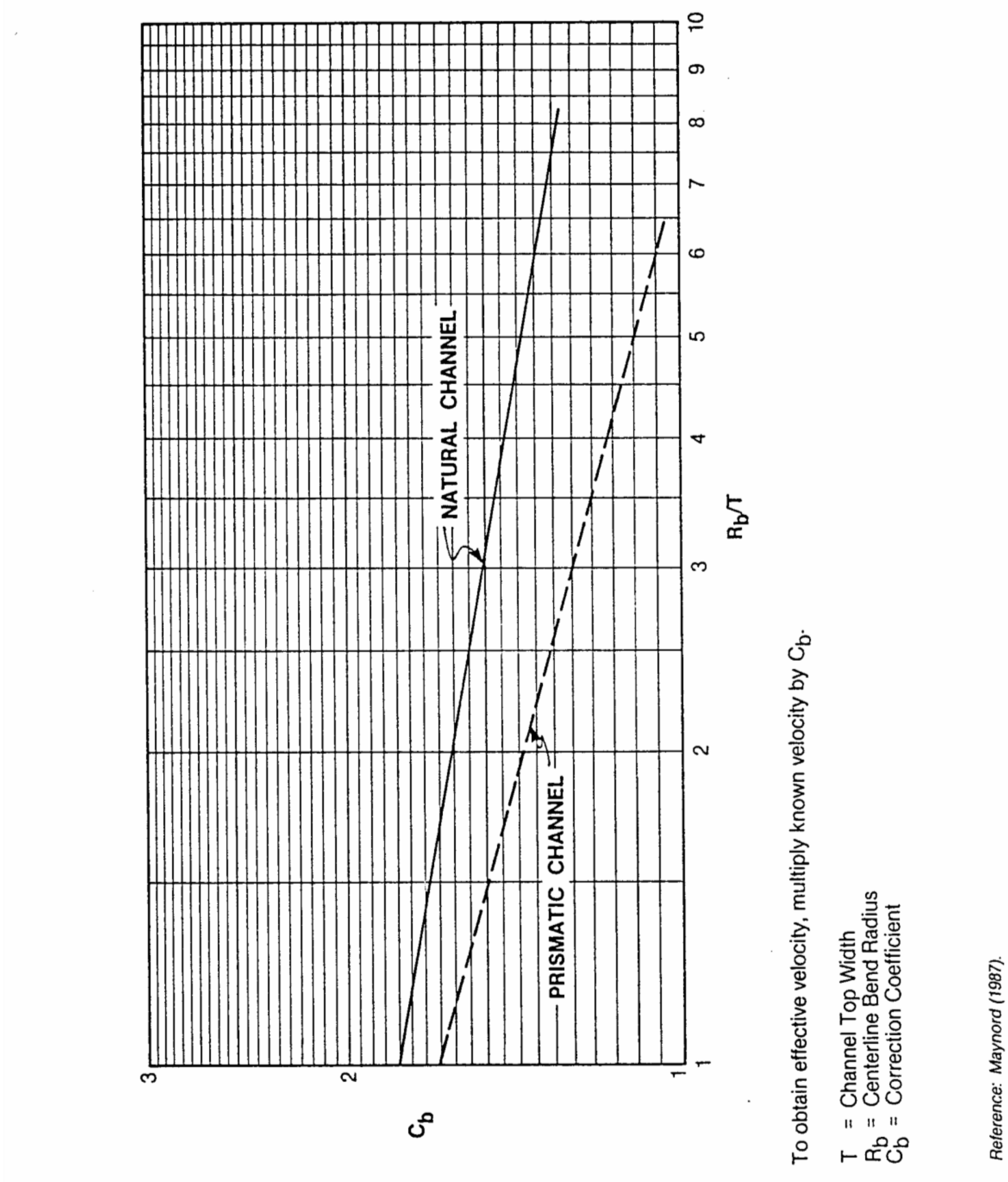


Figure 3.8 Riprap Lining Bend Correction Coefficient

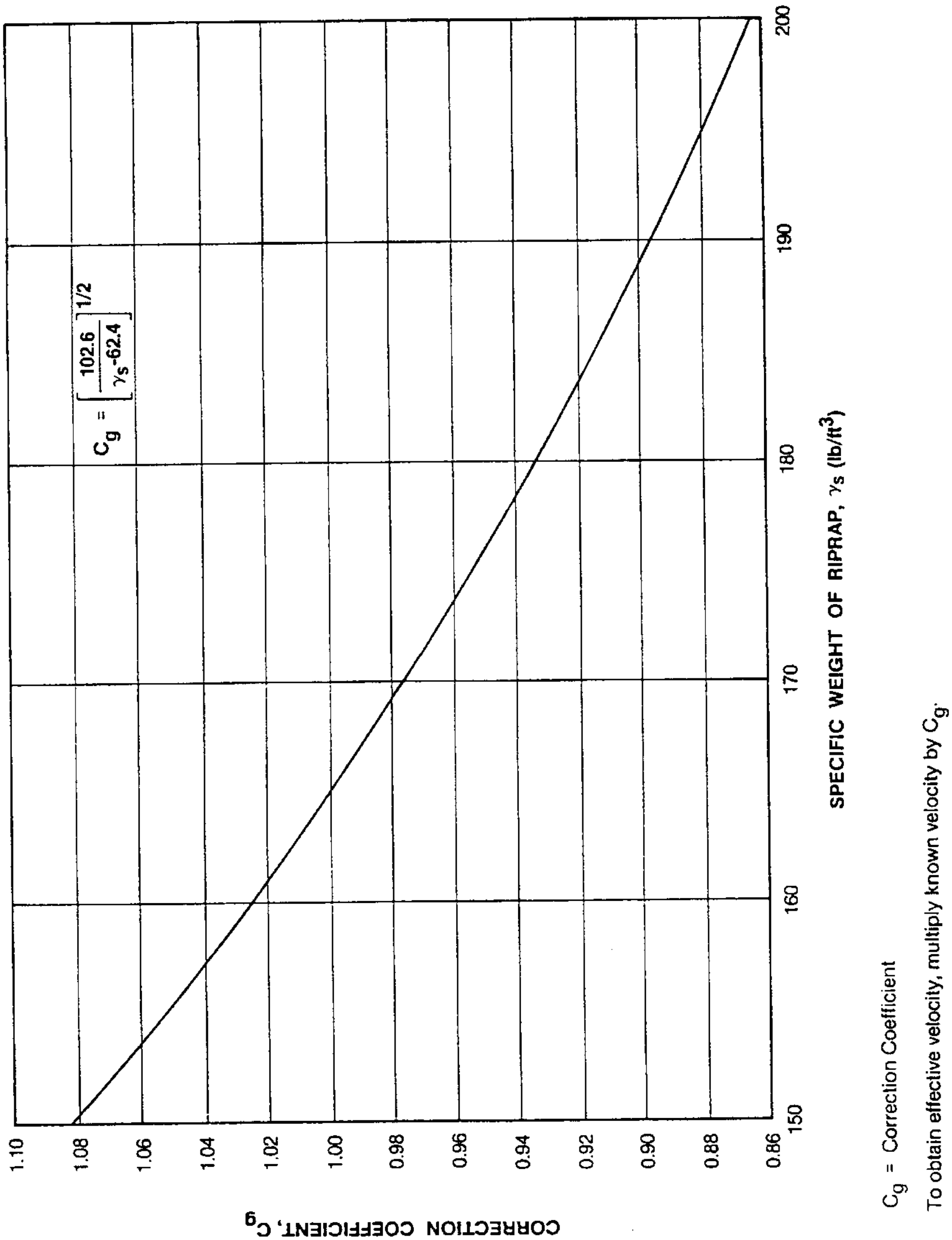
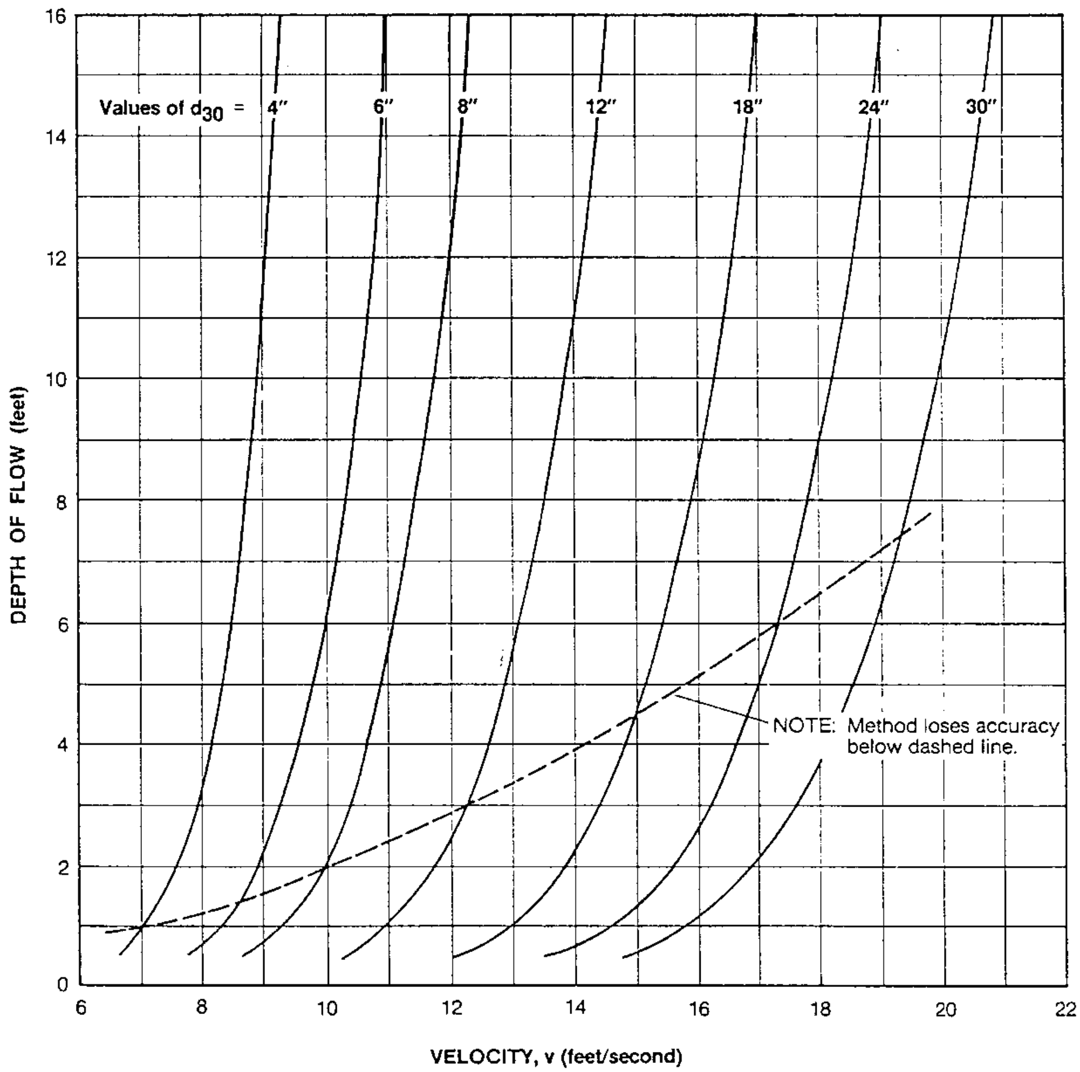


Figure 3.9 Riprap Lining Specific Weight Correction Coefficient
(Source: Nashville Stormwater Management Manual, 1988)



Reference: Reese (1988).

Figure 3.10 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:

$$\mathbf{W = 0.5236\ SW_s\ d^3} \qquad \textbf{(3.14)}$$

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 3.11 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.

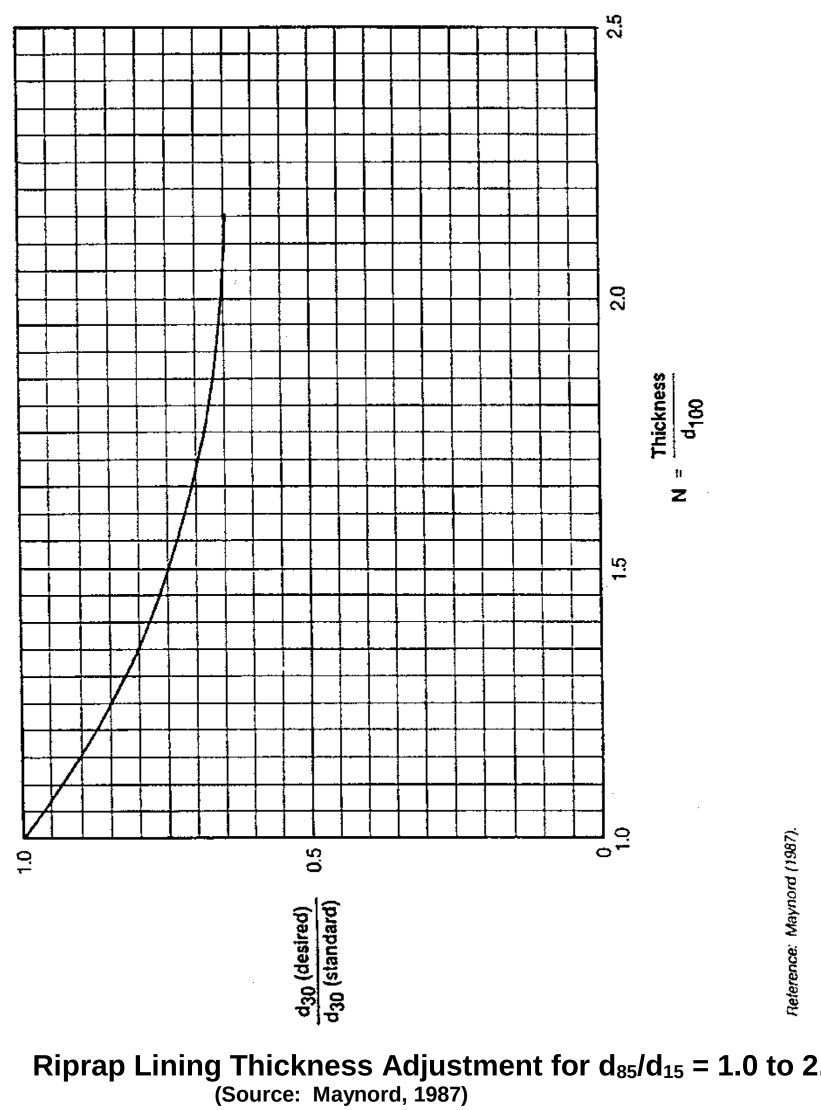


Figure 3.11 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3
(Source: Maynord, 1987)

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:

$$\mathbf{T_o = \Upsilon_w RS} \qquad \textbf{(3.15)}$$

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)

$$\mathbf{T_o' = T_o(1-(\sin^2\Phi/\sin^2\Theta))^{0.5}} \qquad \textbf{(3.16)}$$

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:

$$\mathbf{T_b = 3.15T(r/w)^{-0.5}} \qquad \textbf{(3.17)}$$

where:

T_b = local tractive stress in the bend (lb/ft^2)

T = the greater of T_o or T_o' from Equations 3.15 and 3.16

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:

$$\mathbf{D_{50} = T/0.04(\Upsilon_s-\Upsilon_w)} \qquad \textbf{(3.18)}$$

where:

D_{50} = required average size of riprap stone (ft)

T = the greater of T_o or T_o' from Equations 3.15 and 3.16

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 3.12 through 3.17. Select from smaller side of band at 50% finer gradation.

Step 5 Select riprap gradations table (Figures 3.18 and 3.19) based on riprap thickness selected in Step 4.

Step 6 Select bedding thickness from grain size curves in Figures 3.12 through 3.17, which was used to select the riprap thickness in Step 4. Note: The bedding thicknesses included in Figures 3.12 through 3.17 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:

$$\textbf{GRADATION INDEX: } [D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5 \qquad (3.19)$$

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:

$$D_{15}/d_{85} < 5 < D_{15}/d_{15} < 40 \qquad (3.20)$$

$$D_{50}/d_{50} < 40 \qquad (3.21)$$

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 3.20 and 3.21, from the USACE publication EM1110-2-160.

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:

$$D_{50} = \sqrt{V}/[1.8\sqrt{(2g(\Upsilon_s-\Upsilon_w)/\Upsilon_w)}] \qquad (3.22)$$

Step 2 Select riprap and bedding from Figures 3.12 through 3.17 using D_{50} from Equation 3.22.

Step 3 Select gradations from tables in Figures 3.18 and 3.19.

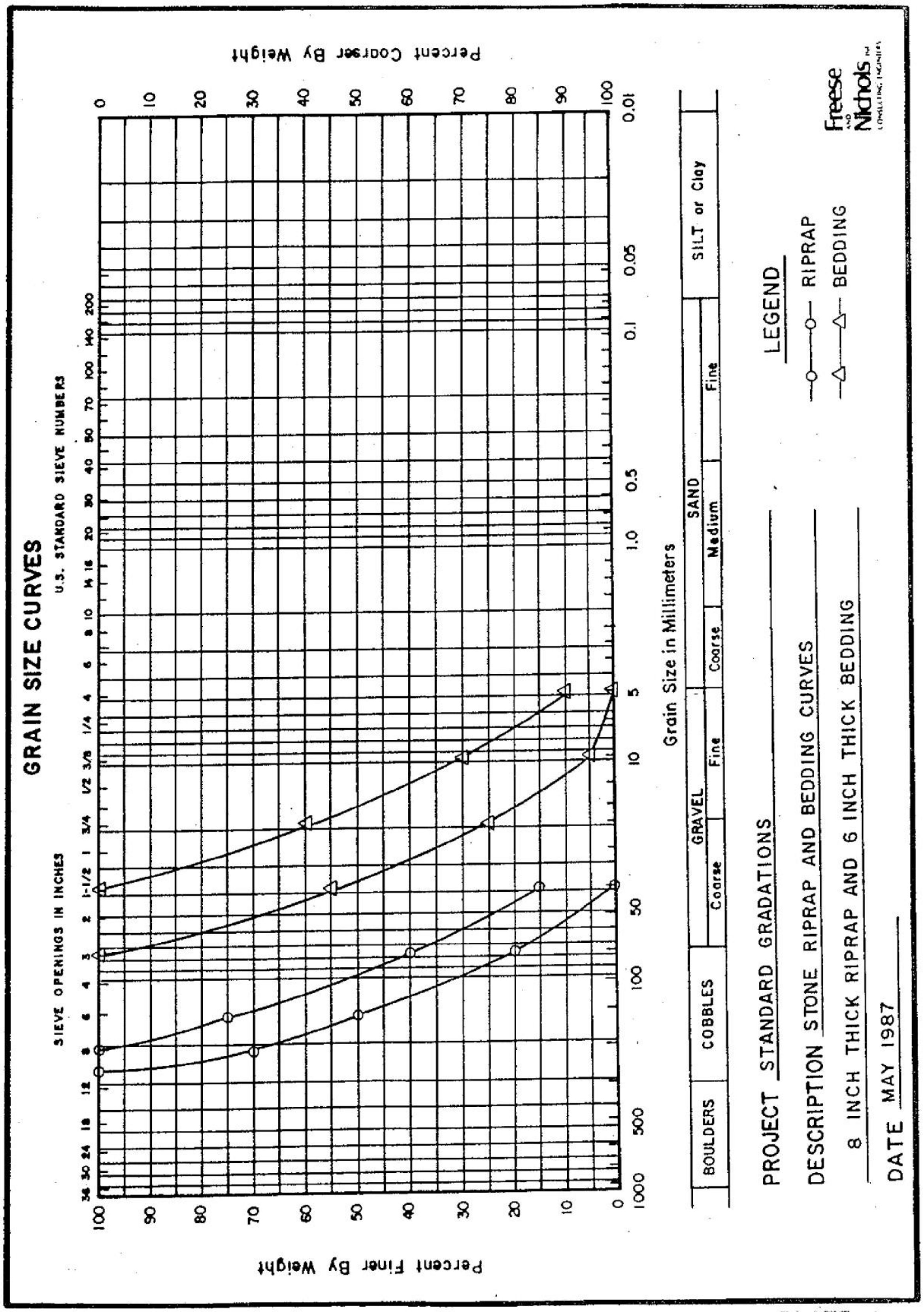


Figure 3.12 Grain Size Curve for 8" Riprap and 6" Bedding

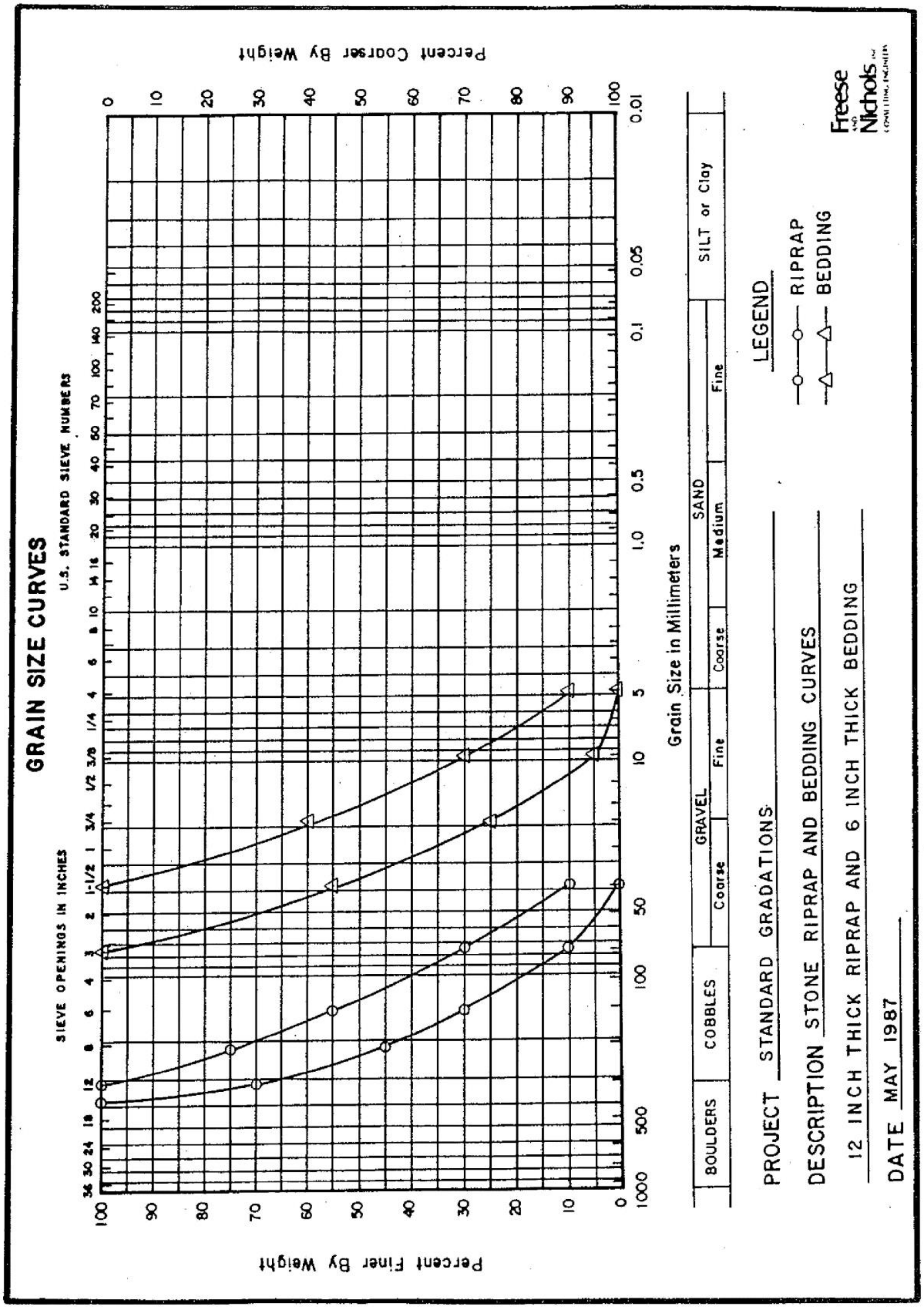


Figure 3.13 Grain Size Curve for 12" Riprap and 6" Bedding

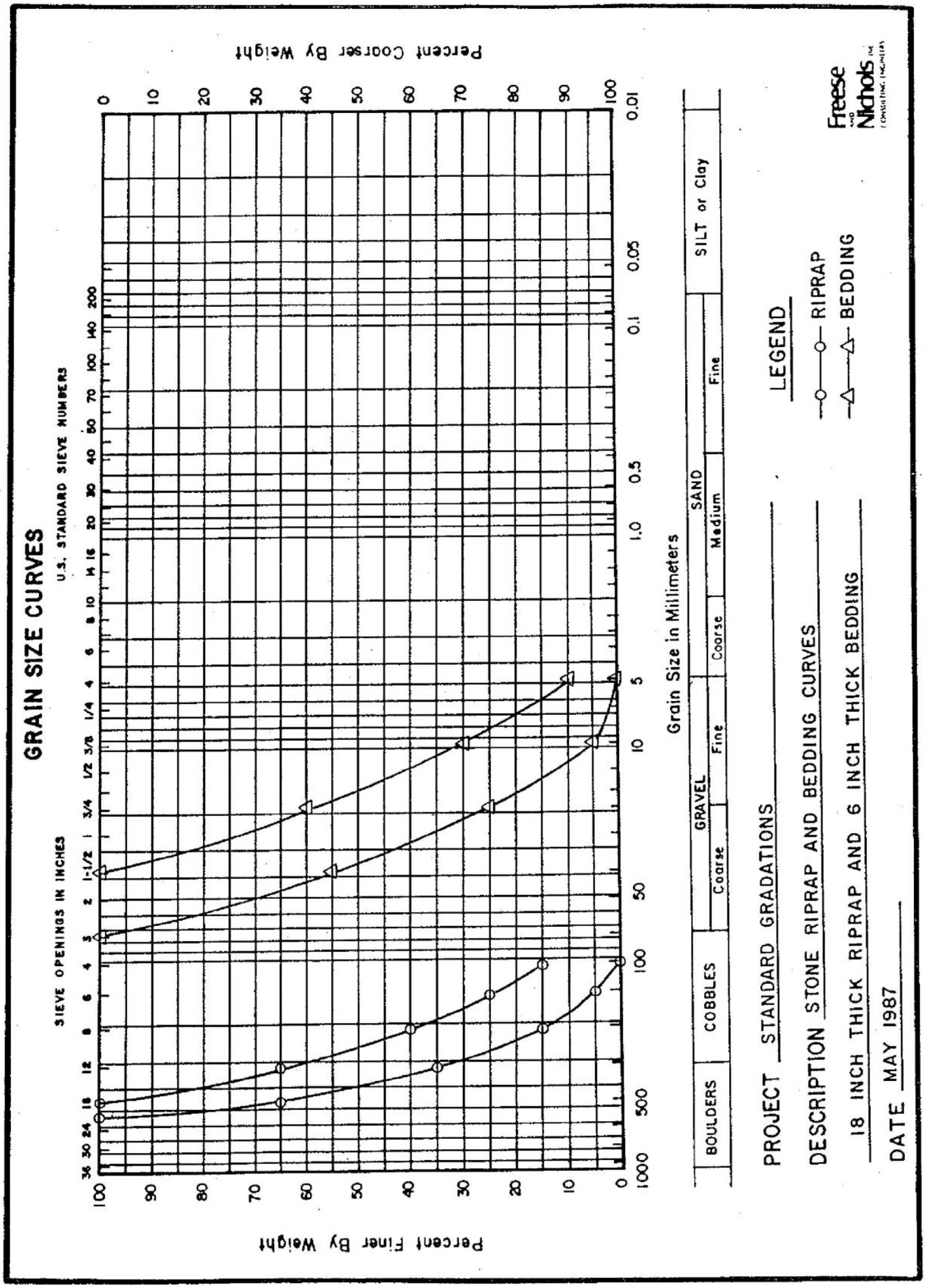


Figure 3.14 Grain Size Curve for 18" Riprap and 6" Bedding

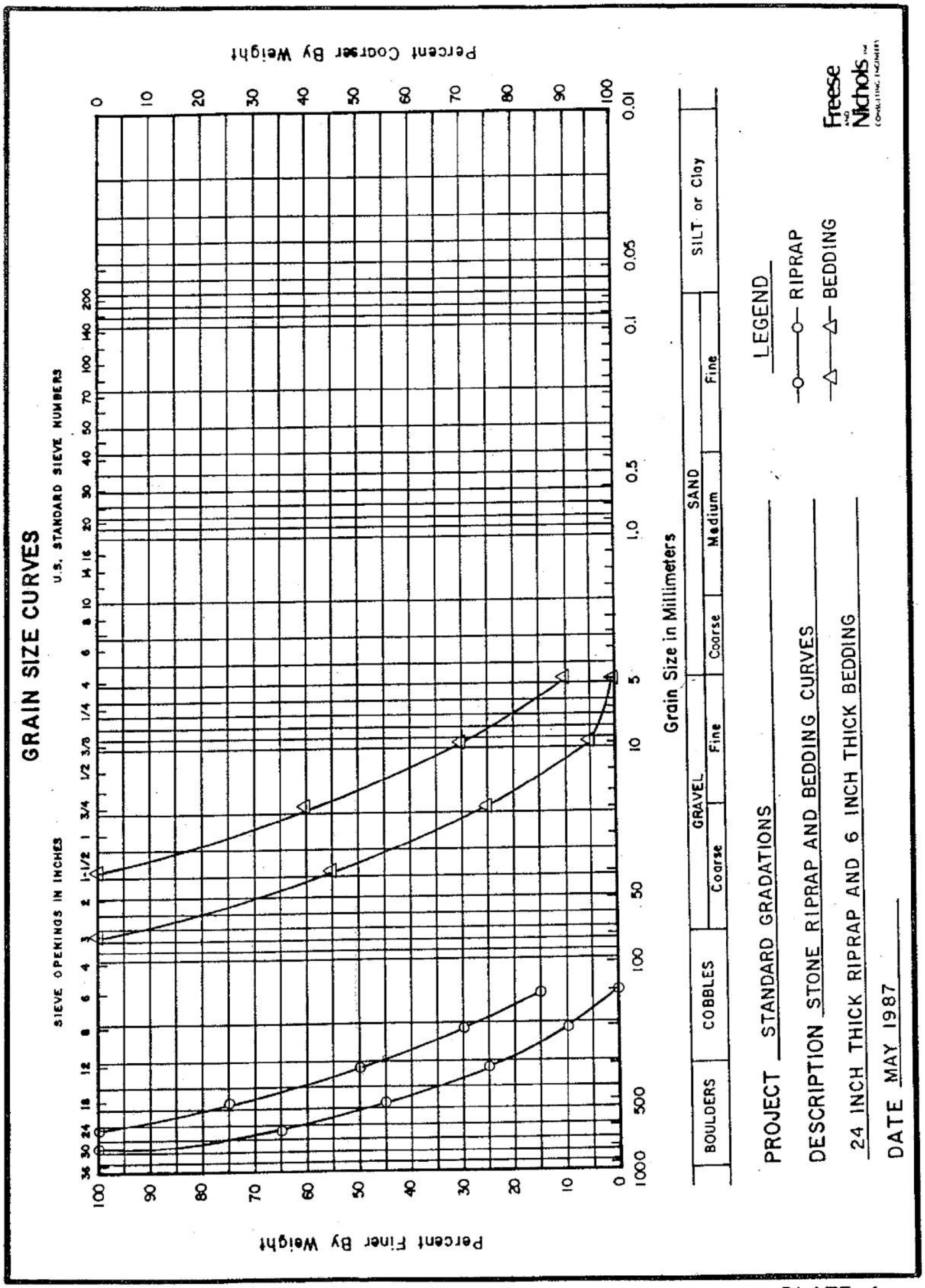


Figure 3.15 Grain Size Curve for 24" Riprap and 6" Bedding

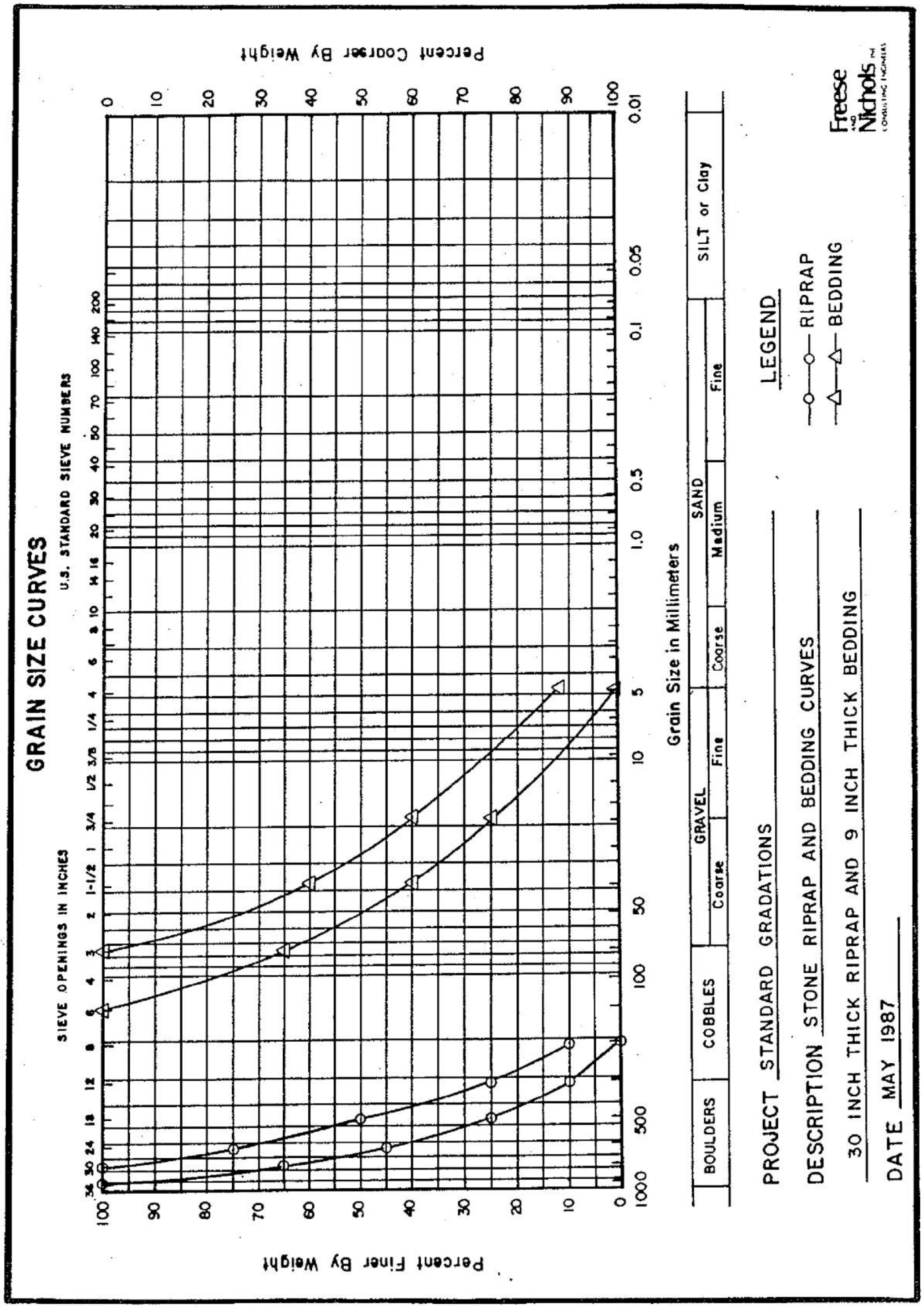


Figure 3.16 Grain Size Curve for 30" Riprap and 9" Bedding

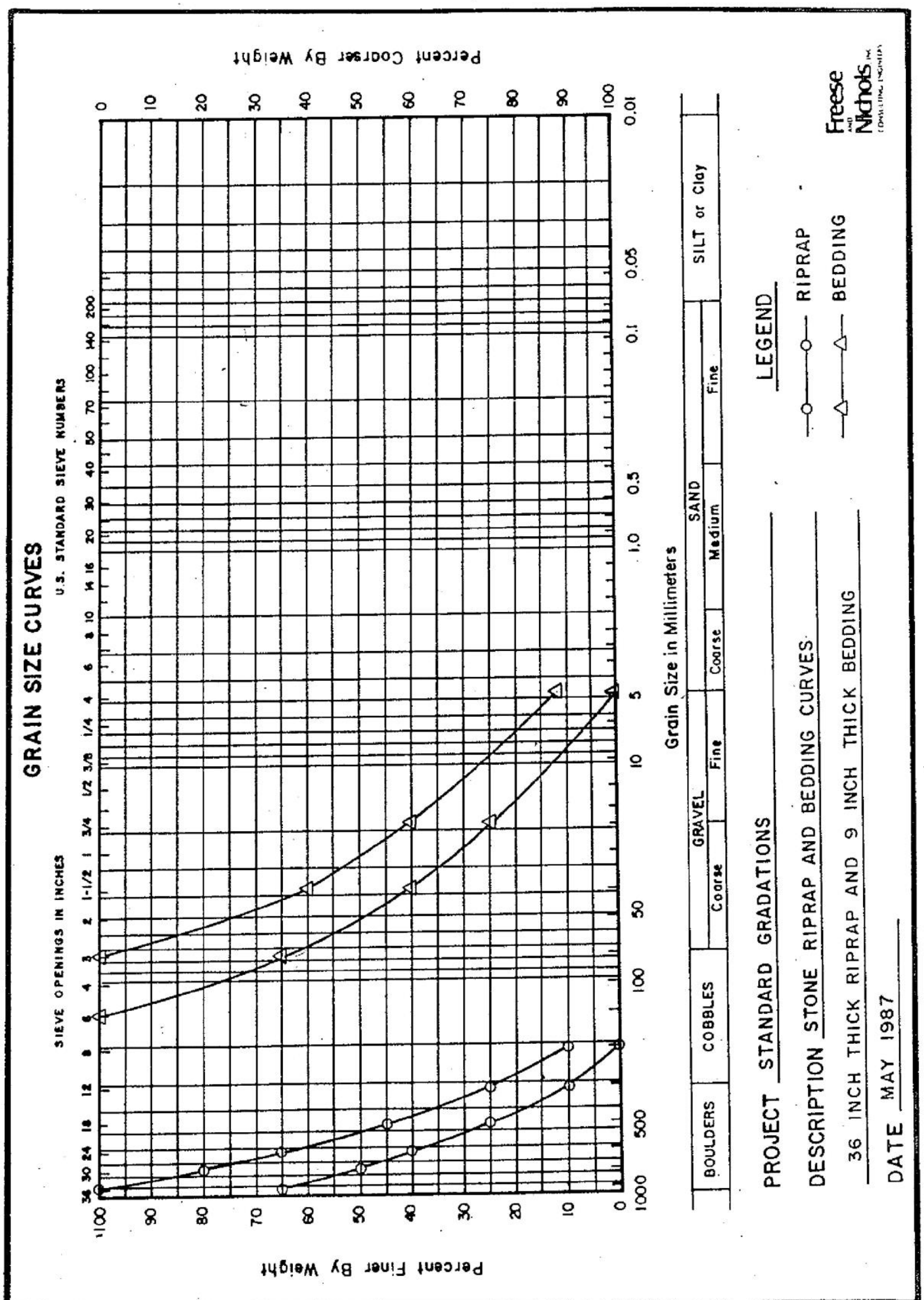


Figure 3.17 Grain Size Curve for 36" Riprap and 9" Bedding

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

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

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 — 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 3.18 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 3.19 Riprap Gradation Tables for 18", 24", 30", and 36" Thickness of Riprap

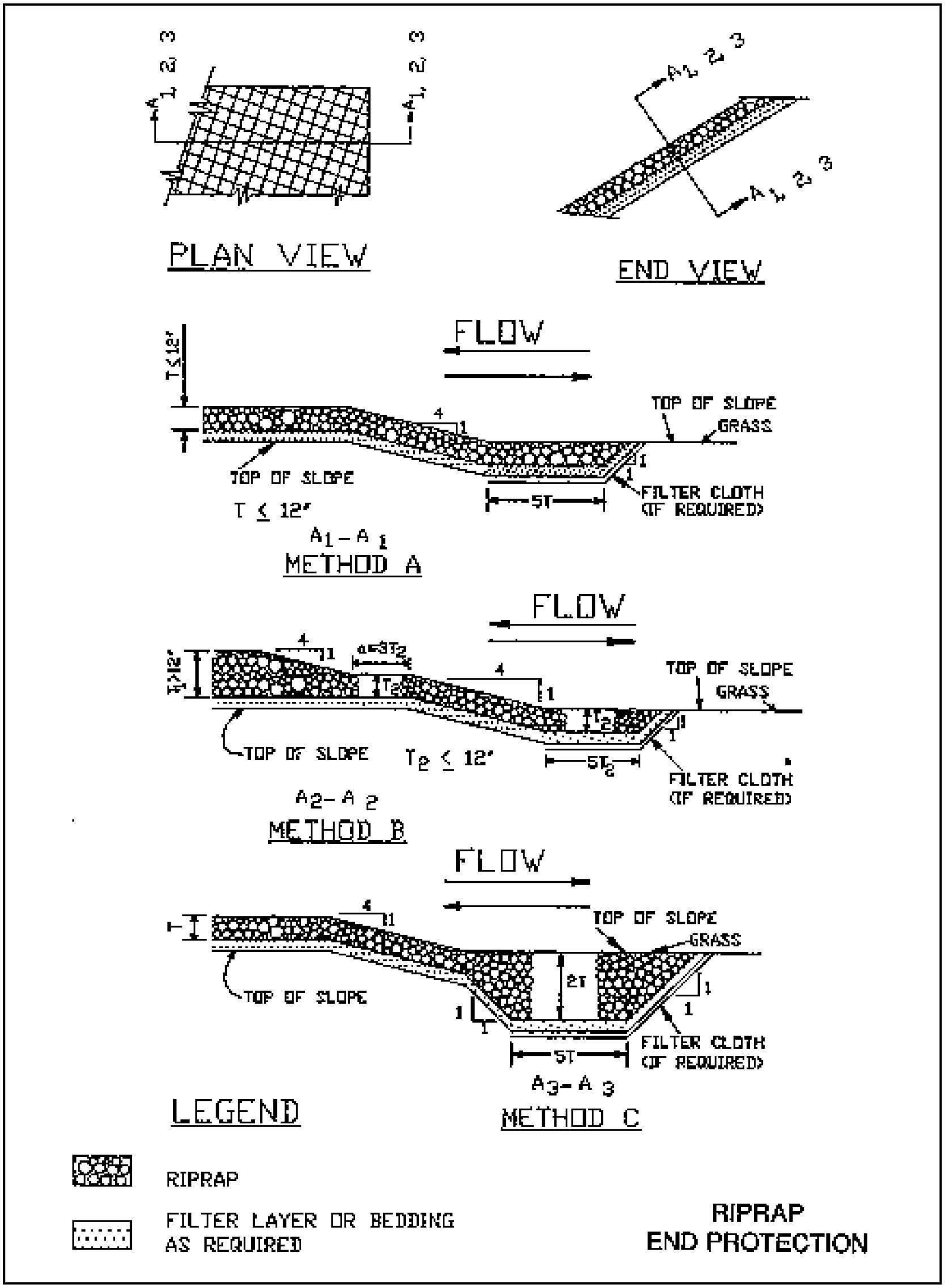


Figure 3.20 Typical Riprap Design Cross Sections

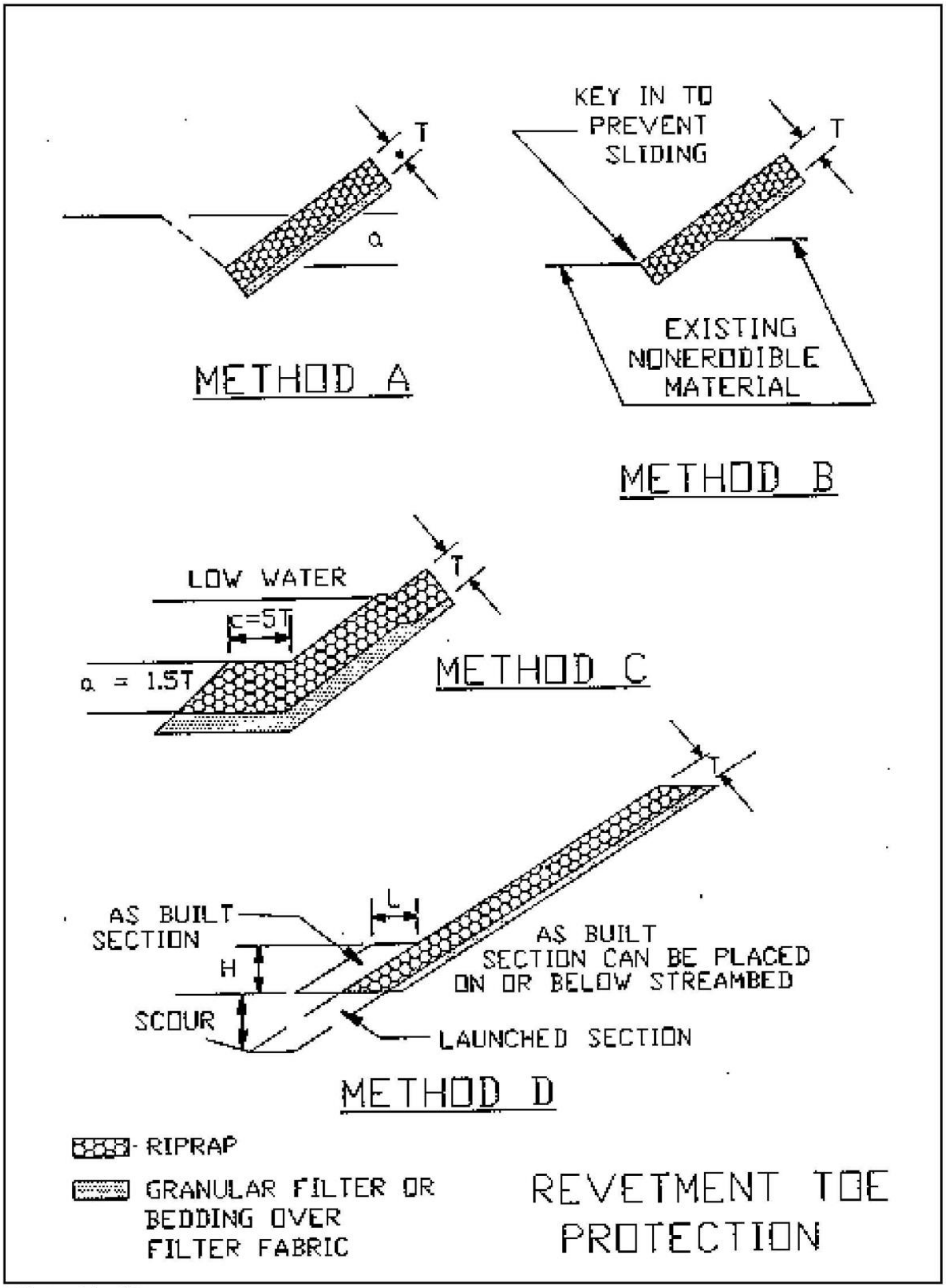


Figure 3.21 Typical Riprap Design for Revetment Toe Protection

3.2.8 Gabion Design

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 3.22), 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.

Figure 3.22 Gabion Baskets Installed for Slope Stabilization along a Stream

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 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.

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.

Primary Design Considerations

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.

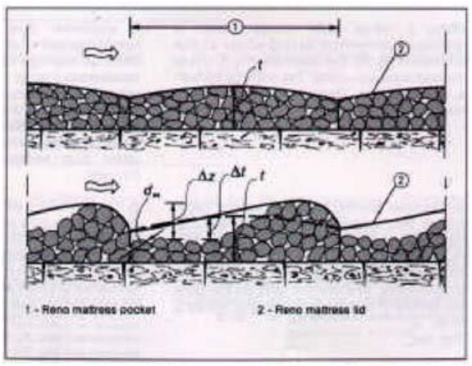
The median stone size for a gabion mattress can be determined from the following equation:

$$\mathbf{d_m = S_f C_s C_v d[(\Upsilon_w/(\Upsilon_s - \Upsilon_w))^{0.5}(V/\sqrt{(gdK_1)})]^{2.5}} \qquad \mathbf{(3.23)}$$

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 3.8)
Υ_w	=	unit weight of water (62.4 lb/ft^3)
Υ_s	=	unit weight of stone (lb/ft^3)

Table 3.8 Values of K_1 for various Side Slopes to be used in Equation 3.23	
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 3.23 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 3.23 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 3.9.

Figure 3.23 Gabion Mattress Showing Deformation of Mattress Pockets under High Velocities

Table 3.9 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 3.9 are compared to Equation 3.22, if V = 11.5, C_s = 0.1, C_v = 1.0, K_1 = 0.71, ,γ_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 3.9. Designers should use Equation 3.23 to take the depth of flow into account. Table 3.9 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:

$$\tau_b = \Upsilon_w S d \tag{3.24}$$

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.,

$$\tau_m = 0.75\ \tau_m \tag{3.25}$$

These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\Upsilon_s - \Upsilon_w)d_m \tag{3.26}$$

with critical shear stress for the banks given as:

$$\tau_s = \tau_c\sqrt{(1-(\sin^2\Theta/0.4304))} \tag{3.27}$$

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.

<u>Stability of Underlying Bed and Bank Material</u>

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.

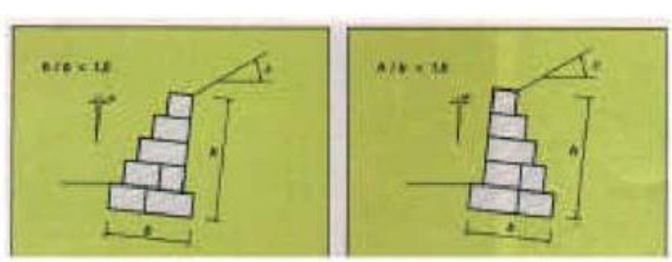


Figure 3.24 Front-step and Rear-step Gabion Layout

The velocity at the mattress/filter interface, V_b, is estimated to be

$$V_b = 1.486\sqrt{S}(d_m/2)^{2/3}/n_f \qquad (3.28)$$

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:

$$V_f = 1.486S\sqrt{V_a}(d_m/2)^{2/3}/n_f \qquad (3.29)$$

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 5.28. 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.

3.2.9 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 3.3:

- 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.
- 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.

From Table 3.3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.

Assume an n value of 0.035 and find the value of vR from Figure 3.2, vR = 5.4

- Use Equation 3.9 to calculate the value of R: R = 5.4/4 = 1.35 ft
- Use Equation 3.10 to calculate the value of vR:
  $vR = [1.49\,(1.35)^{5/3}\,(0.015)^{1/2}]/0.035 = 8.60$

- 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 3.2)	R (Eq. 3.9)	vR (Eq. 3.10)
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.

- Use Figure 3.4 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 3.1)

Fr = 0.76 (Equation 3.8)

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 3.6):

$A = (b + zd)\,d = [10 + (3)\,(1)]\,(1) = 13.0$ 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 3.2 for retardance Class C (n = 0.047)
- Use Figure 3.3 or Equation 3.1 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. 5.2)	Velocity (Eq. 5.1)
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

- 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 3.8)
  n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (Equation 3.8)

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.

- Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.
- Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 3.8, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.
- Determine the correction coefficient for the specific weight of 170 lbs from Figure 3.9 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.
- Determine minimum d_{30} from Figure 3.10 or Equation 3.13 as about 10 inches.
- Use a gradation with a minimum d_{30} size of 10 inches.
- *(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 3.11, 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).
- 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 3.7.

3.2.10 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.

3.2.11 Rectangular, Triangular and Trapezoidal Open Channel Design

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.

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 1.2) 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.

Instructions for Rectangular and Trapezoidal Figures

Figures such as Figure 3.25 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.

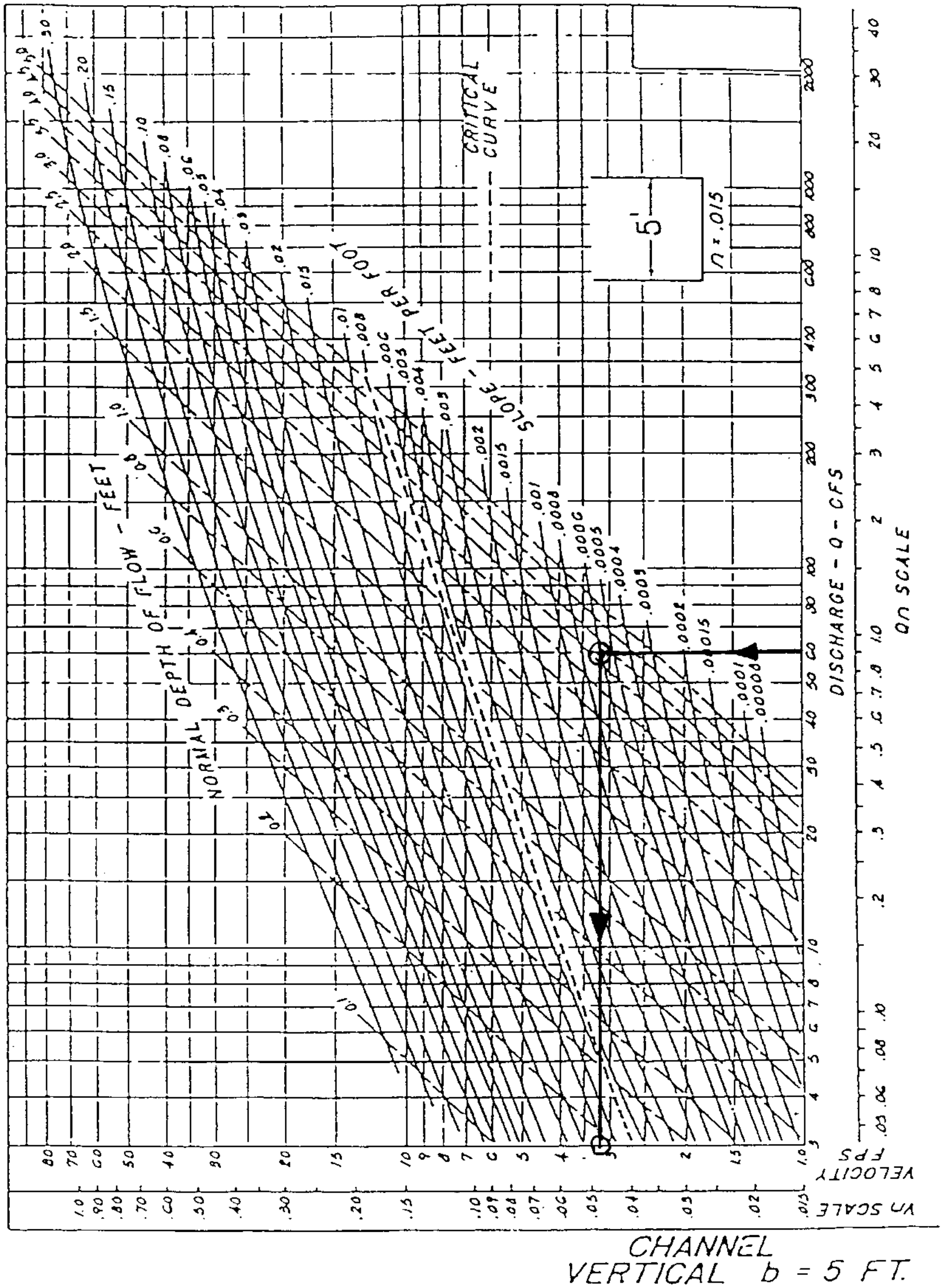
Example Design Problem 1

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 *Section 3.2.11*, select the rectangular figure for a 5-ft width (Figure 3.25).
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 3.25 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 3.26).
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.

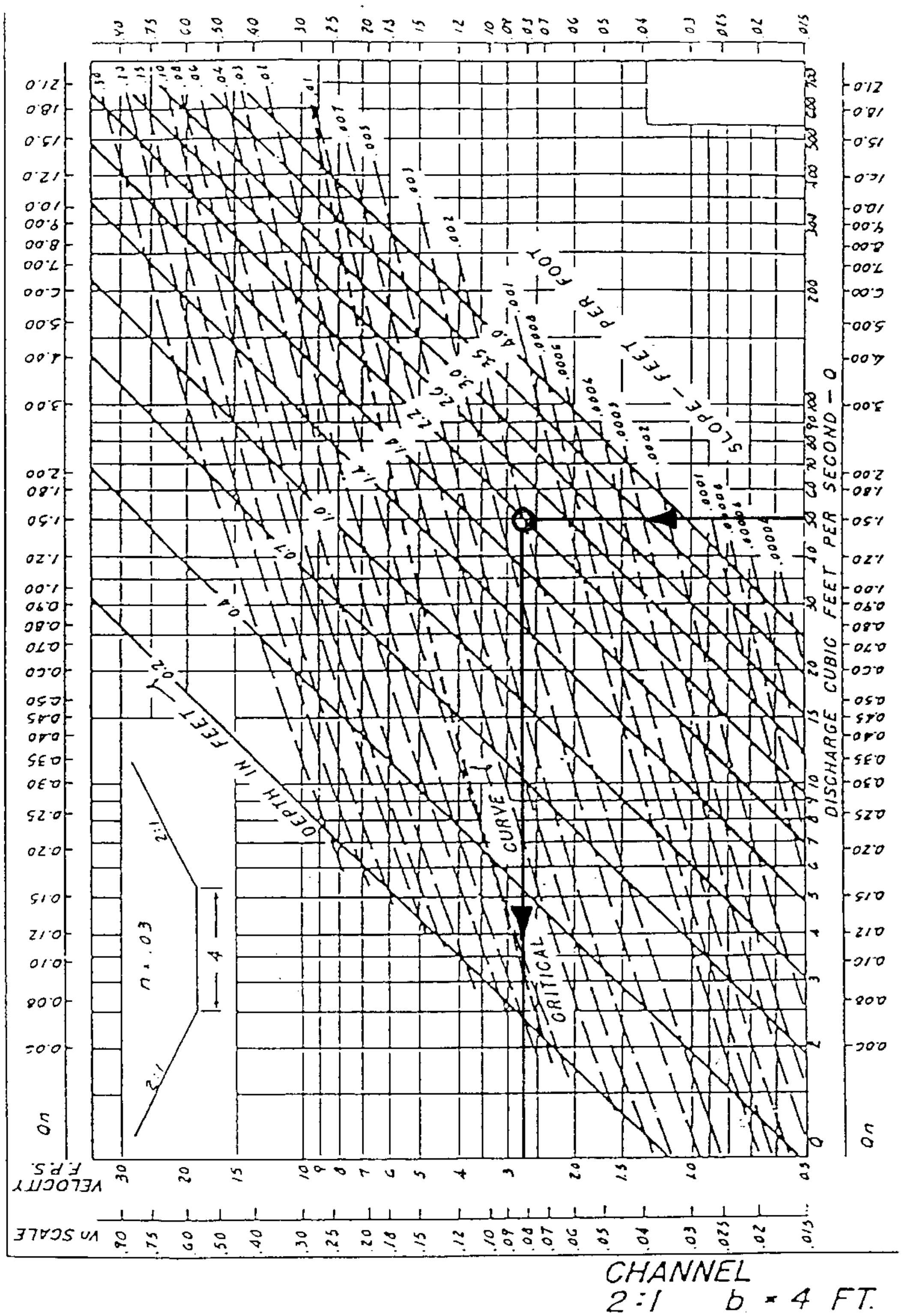
Example Design Problem 3

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

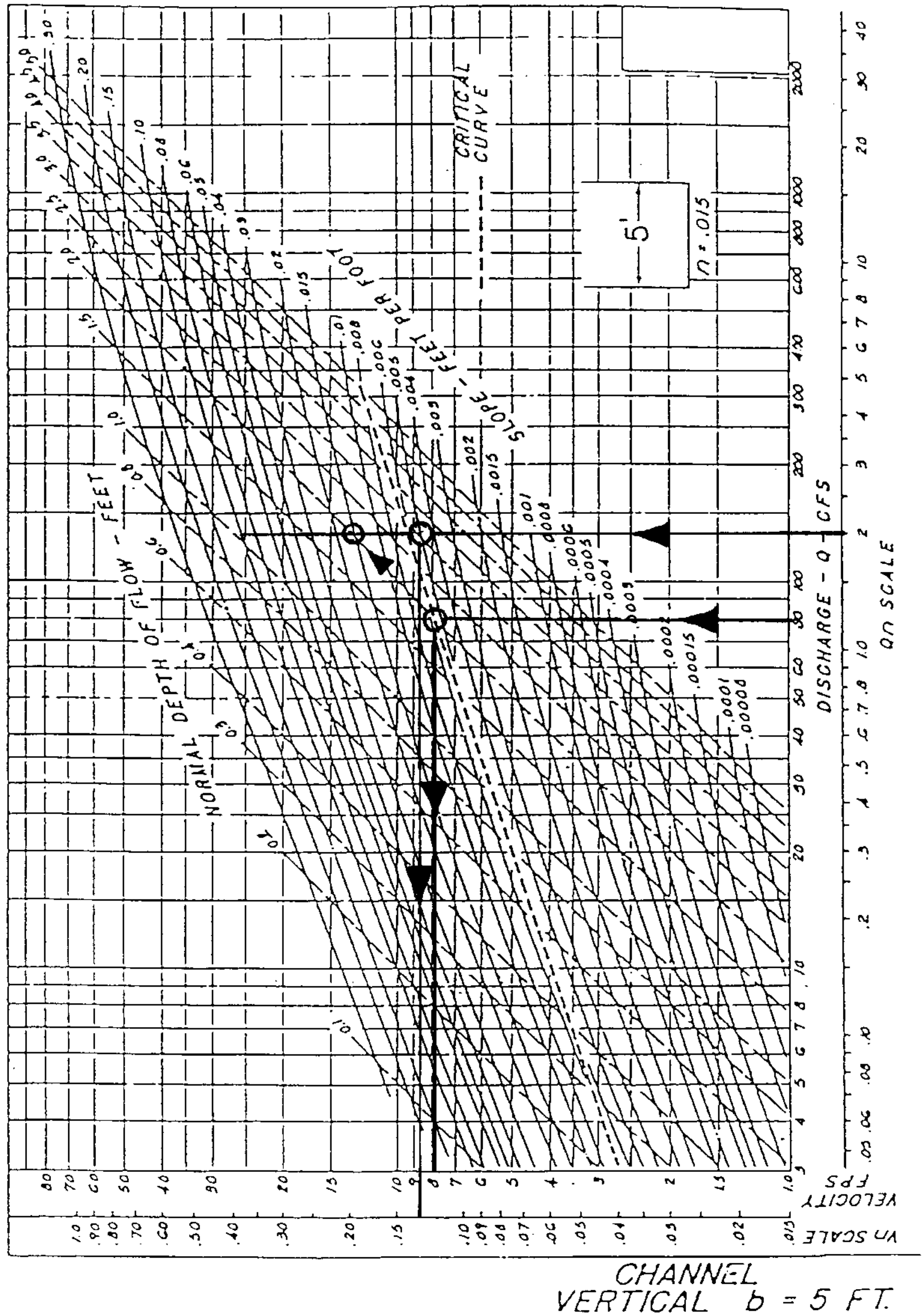
Procedure:

1. Select the rectangular figure for a 5 ft width (Figure 3.27).
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 3.14, 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 3.26 Example Nomograph #2



Source: Federal Highway Administration

Figure 3.27 Example Nomograph #3

3.2.12 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 *Section 3.2.11*. 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.

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 3.6) and the product V times R is shown in Figure 3.2. As indicated in Table 3.6, 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 3.6 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.

Instructions for Grassed Channel Figures

The grassed channel figures like those in Figure 3.12 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 3.2 and 3.3. 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 3.2 and 3.3), 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. Select figure for 4:1 side slopes (see Figure 3.28).
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 3.3.

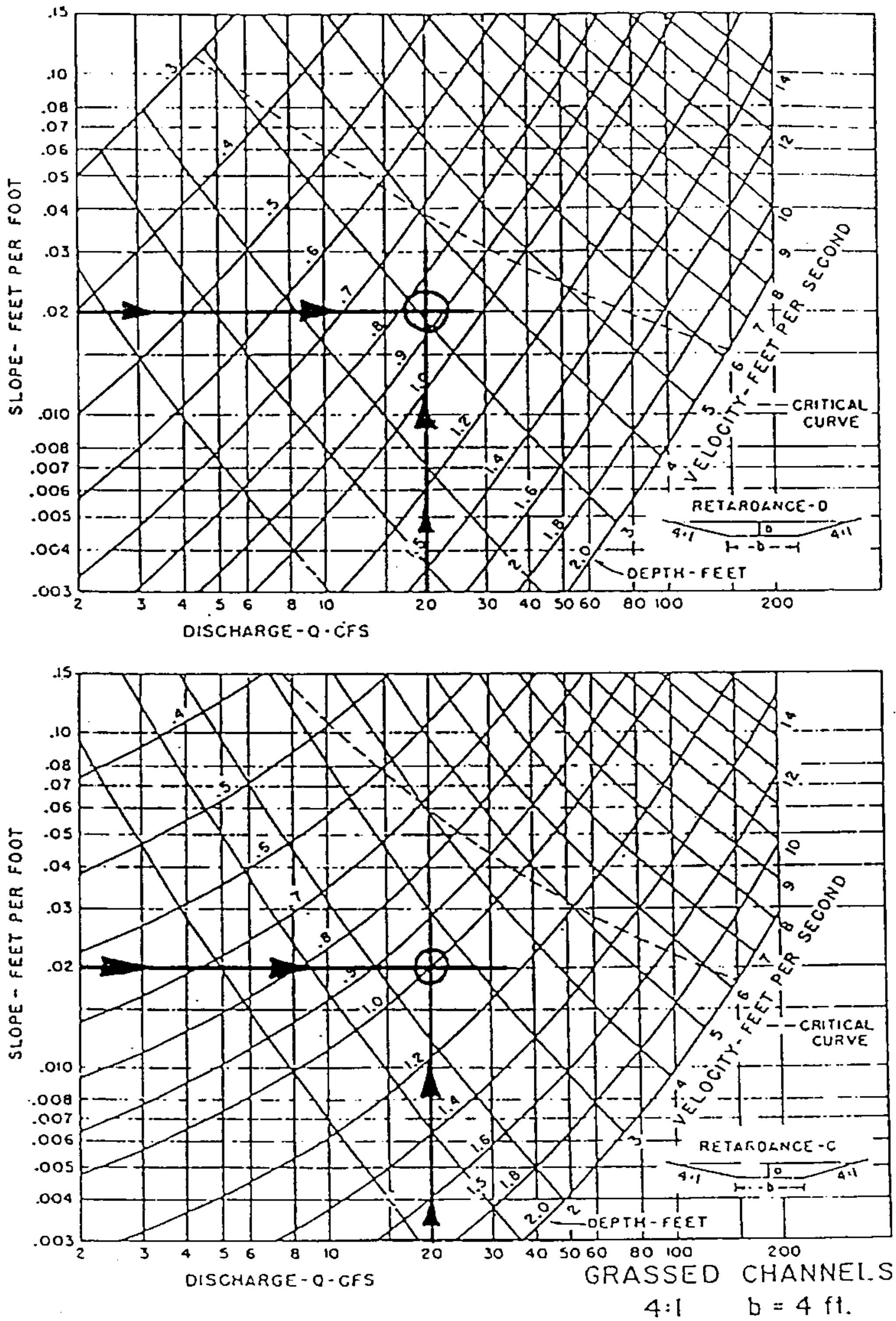
Example Design Problem 2

Given: The channel and discharge of Example 1.

Find: The maximum grade on which the 20 cfs could safely be carried

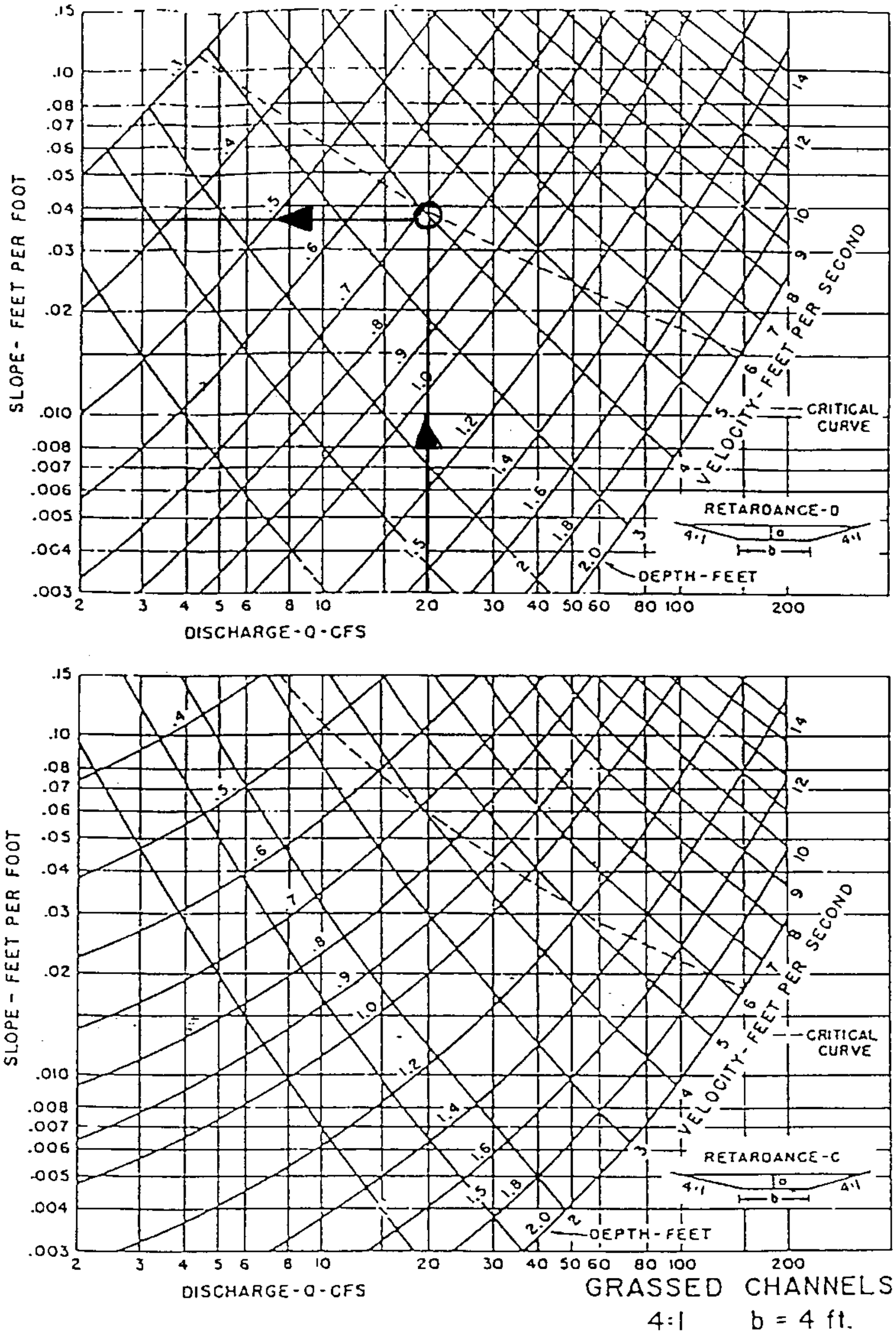
Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 3.3). On the upper graph of Figure 3.29 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 3.28 Example Nomograph #4



Source: Federal Highway Administration

Figure 3.29 Example Nomograph #5

3.3 Culvert Design

3.3.1 Overview

A *culvert* is a short, closed (covered) conduit that conveys stormwater 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).

3.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 3.10 will be used. These symbols were selected because of their wide use.

Table 3.10 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

3.3.3 Design Considerations

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following list of design recommendations should be considered for all culvert designs as applicable. Refer to *Section 3.6.3 of the Criteria Manual* or the local review authority for design criteria details.

- **Frequency Flood**
- **Velocity Limitations**
- **Buoyancy Protection**
    - Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.
- **Length and Slope**
- **Debris Control**
    - In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled Debris Control Structures be consulted.
- **Headwater Limitations**
- **Tailwater Considerations**
- **Storage**
- **Culvert Inlets**
    - Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient Ke, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 3.11.
- **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.
- **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.
- **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.
- **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 3.12 gives recommended Manning's n values for different materials.

- **Culvert Skews**
    - Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.
- **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.
- **Outlet Protection**
    - See *Section 2.2* for information on the design of outlet protection.
- **Erosion and Sediment Control**
- **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.
- **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.

Table 3.11 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, 33.7º or 45º 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, 33.7º or 45º 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 30º to 75º 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 10º or 25º 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 3.12 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.

3.3.4 Design Procedures

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.

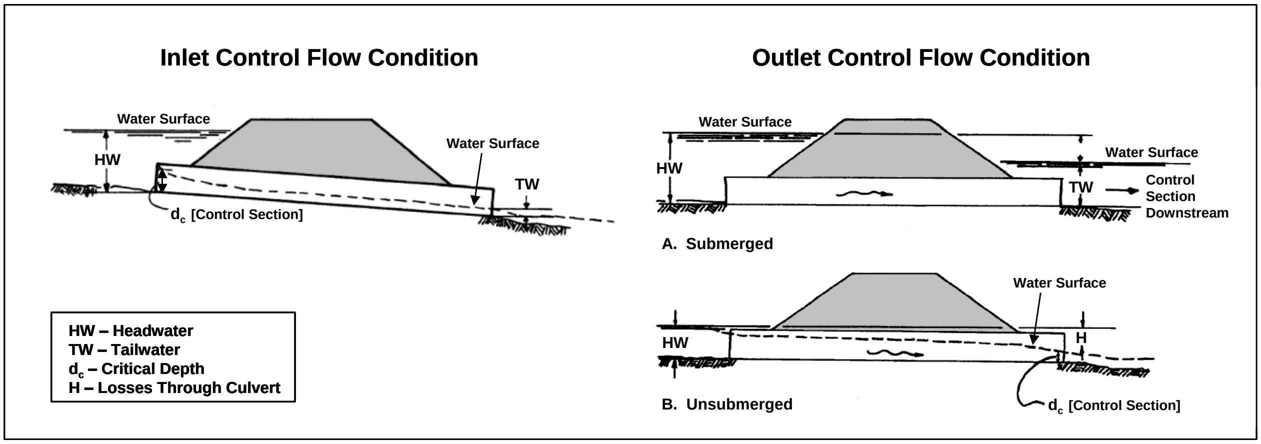


Figure 3.30 Culvert Flow Conditions
(Adapted from: HDS-5, 2001)

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.

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.

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 3.13 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 3.14. Table 3.14 is arranged in the same order as the design nomographs shown later in this section, 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 3.14, 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 3.13 Inlet Control Design Equations		
Unsubmerged*		
Form (1)	$$\frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{K_u Q}{AD^{0.5}}\right)^M - 0.5S \;***$$	**(3.30)**
Form (2)	$$\frac{HW_i}{D} = K\left(\frac{K_u Q}{AD^{0.5}}\right)^M$$	**(3.31)**
Submerged**		
	$$\frac{HW_i}{D} = c\left(\frac{K_u Q}{AD^{0.5}}\right)^2 + Y - 0.5S \;***$$	**(3.32)**
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 3.14	
K_u	1.811 SI (1.0 English)	
* Equations 3.30 and 3.31 (unsubmerged) apply to about $Q/AD^{0.5}$ = 1.93 (3.5 English)		
** Equation 3.32 (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 3.14 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, 45º bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7º bevels		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30º to 75º wingwall flares	1	.026	1.0	.0347	.81	56
		2	90º and 15º wingwall flares		.061	.75	.0400	.80	56
		3	0º wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45º wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18º to 33.7º wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90º headwall w/ 3/4” chamfers	2	.515	.667	.0375	.79	8
		2	90º headwall w/ 45º bevels		.495	.667	.0314	.82	
		3	90º headwall w/ 33.7º bevels		.486	.667	.0252	.865	

Table 3.14 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	
11	Rectangular	1	3/4” chamfers; 45º skewed	2	.545	.667	.0451	.73	8
	Box		headwall		.533	.667	.0425	.705	
		2	3/4” chamfers; 30º skewed		.522	.667	.0402	.68	
			headwall		.498	.667	.0327	.75	
		3	3/4” chamfers; 15º skewed						
			headwall						
		4	45º bevels; 10º-45º skewed						
			headwall						
12	Rectangular	1	45º non-offset wingwall flares	2	.497	.667	.0339	.803	8
	Box 3/4”	2	18.4º non-offset wingwall flares		.493	.667	.0361	.806	
	chamfers		18.4º non-offset wingwall flares		.495	.667	.0386	.71	
		3	30º skewed barrel						
13	Rectangular	1	45º wingwall flares - offset	2	.497	.667	.0302	.835	8
	Box Top	2	33.7º wingwall flares - offset		.495	.667	.0252	.881	
	Bevels	3	18.4º wingwall flares - offset		.493	.667	.0227	.887	
16-19	CM Boxes	2	90º 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	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
	Ellipse	2	Groove end w/ headwall		.0018	2.5	.0292	.74	
	Concrete	3	Groove end projecting		.0045	2.0	.0317	.69	
30	Vertical	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
	Ellipse	2	Groove end w/ headwall		.0018	2.5	.0292	.74	
	Concrete	3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch	1	90º headwall	1	.0083	2.0	.0379	.69	57
	18” Corner	2	Mitered to slope		.0300	1.0	.0463	.75	
	Radius CM	3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch	1	Projecting	1	.0300	1.5	.0496	.57	56
	18” Corner	2	No Bevels		.0088	2.0	.0368	.68	
	Radius CM	3	33.7º Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch	1	Projecting	1	.0300	1.5	.0496	.57	56
	31” Corner	2	No Bevels		.0088	2.0	.0368	.68	
	Radius CM	3	33.7º Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90º 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	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
	Inlet Face	2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge		.547	.80	.0598	.75	
			projecting						
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular	1	Side tapered-less favorable	2	.56	.667	.0446	.85	3
	Concrete		edges		.56	.667	.0378	.87	
		2	Side tapered-more favorable						
			edges						

Table 3.14 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	
59	Rectangular Concrete	1 2	Slope tapered-less favorable edges Slope tapered-more favorable edges	2	.50 .50	.667 .667	.0446 .0378	.65 .71	3

* 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.

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 3.31 (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.

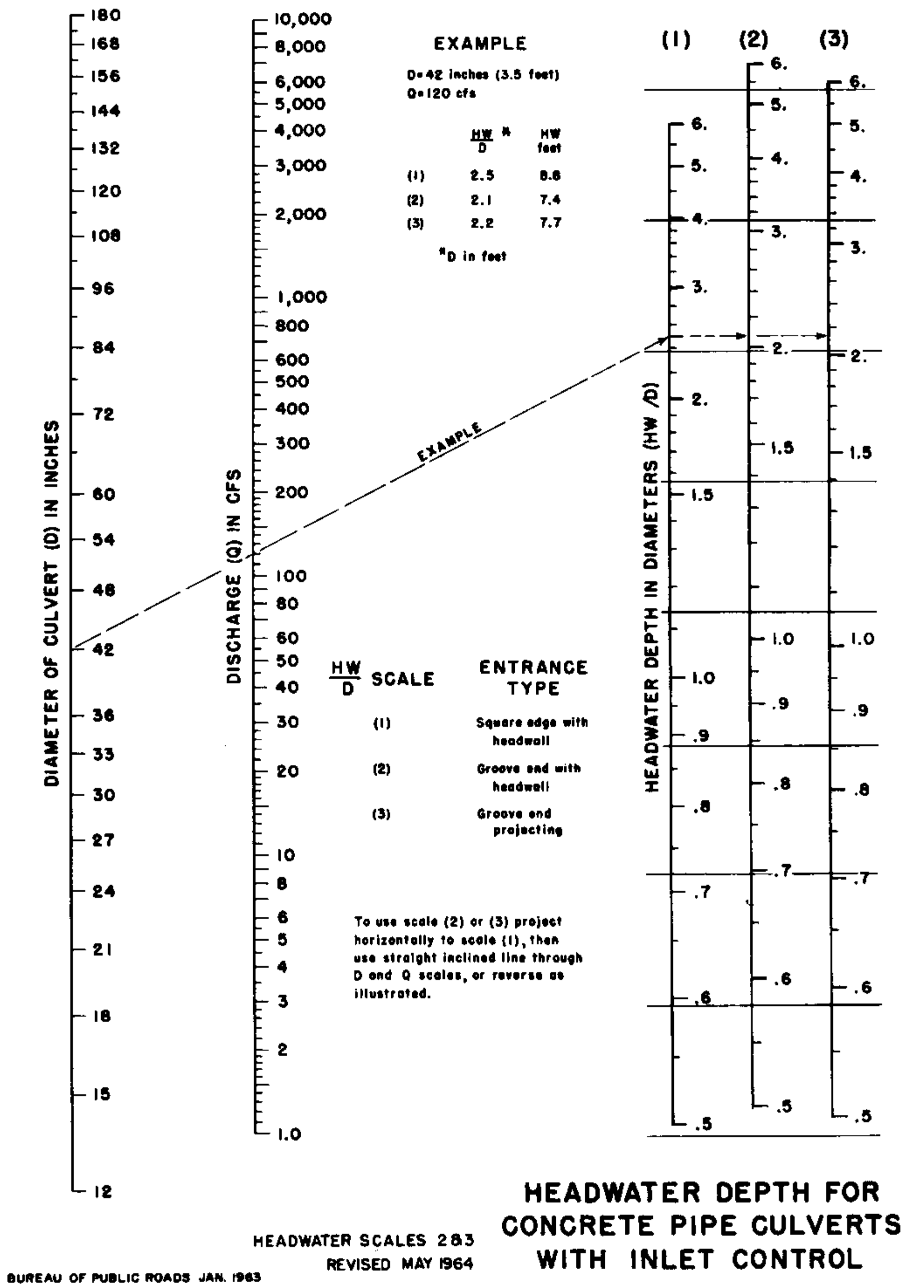


Figure 3.31a Headwater Depth for Concrete Pipe Culvert with Inlet Control

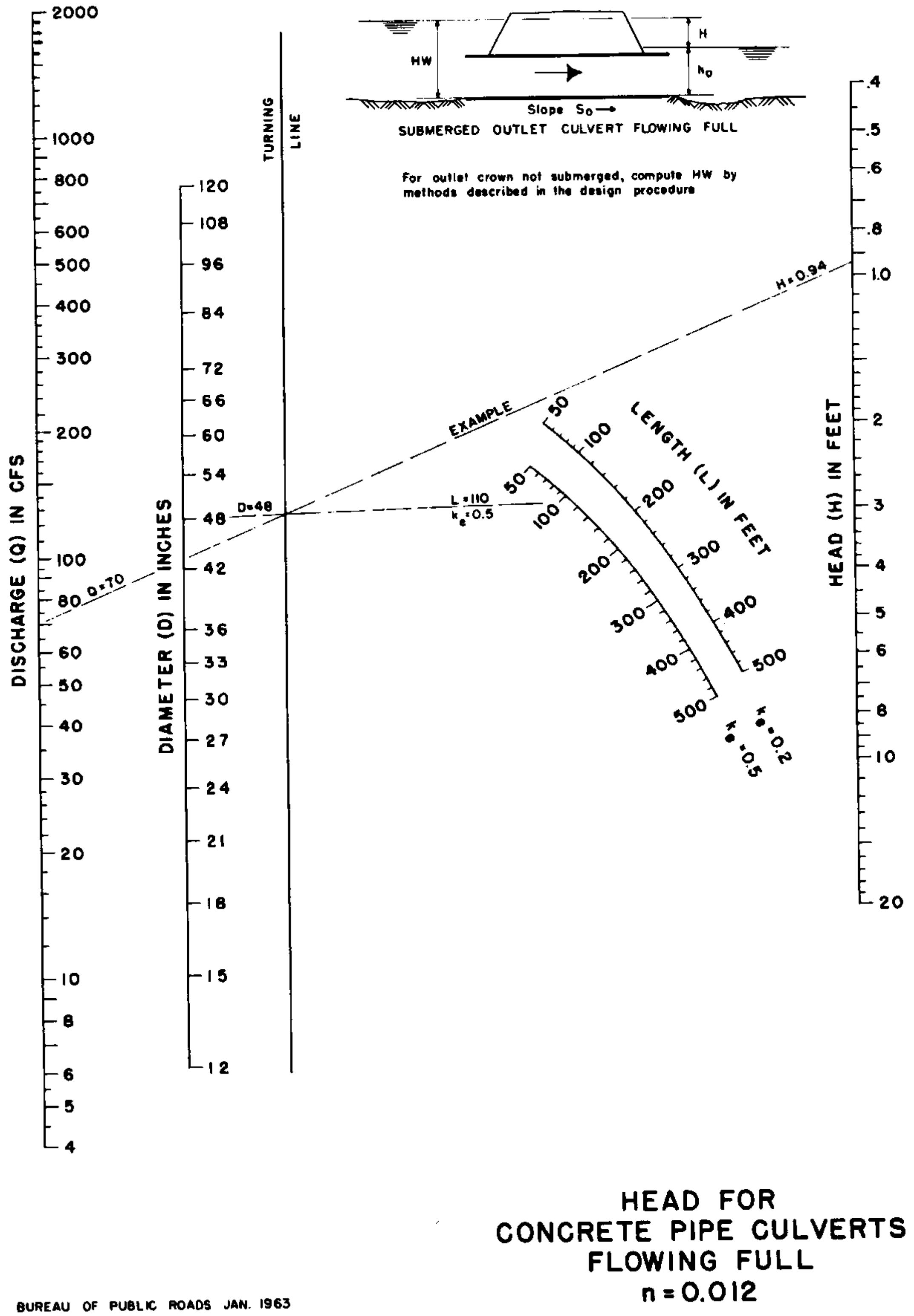


Figure 3.31b Head for Concrete Pipe Culverts Flowing Full

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:

$$\mathbf{HW = H + h_o - LS} \qquad \mathbf{(3.33)}$$

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 4.0* for appropriate energy dissipation designs. Energy dissipation designs may affect the outlet hydraulics of the culvert.

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 3.34 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d L(HW)^{1.5}} \qquad \mathbf{(3.34)}$$

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 3.32 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.

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 *Section 3.3.7* 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.

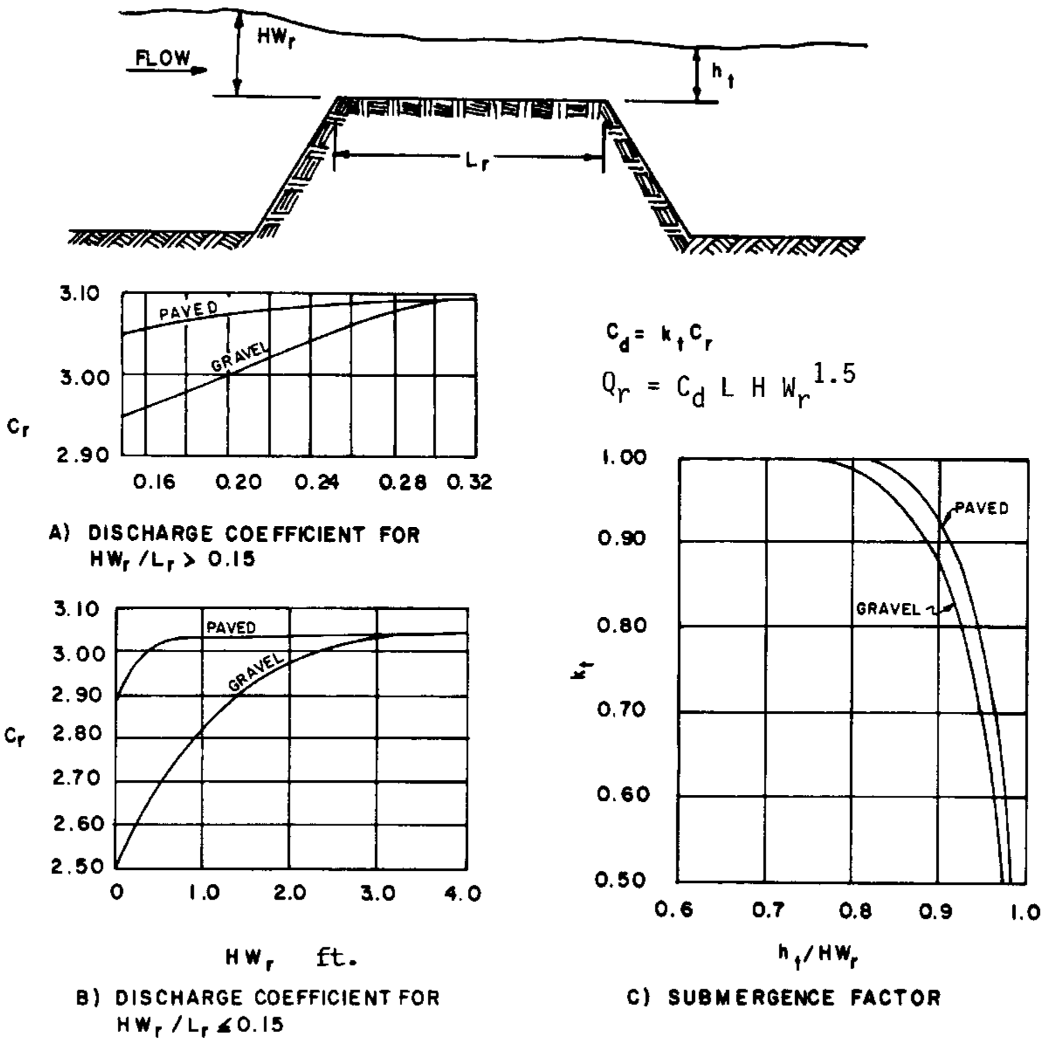


Figure 3.32 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 2001)

3.3.5 Culvert Design Example

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

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.

<u>Input Data</u>

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

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 3.31a), 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 3.31b.

   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 1.19(b).

   Therefore: h_o = 3.5 ft

   The headwater depth for outlet control is:

   HW = H + h_o - LS = 2.8 + 3.5 - (100) x (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

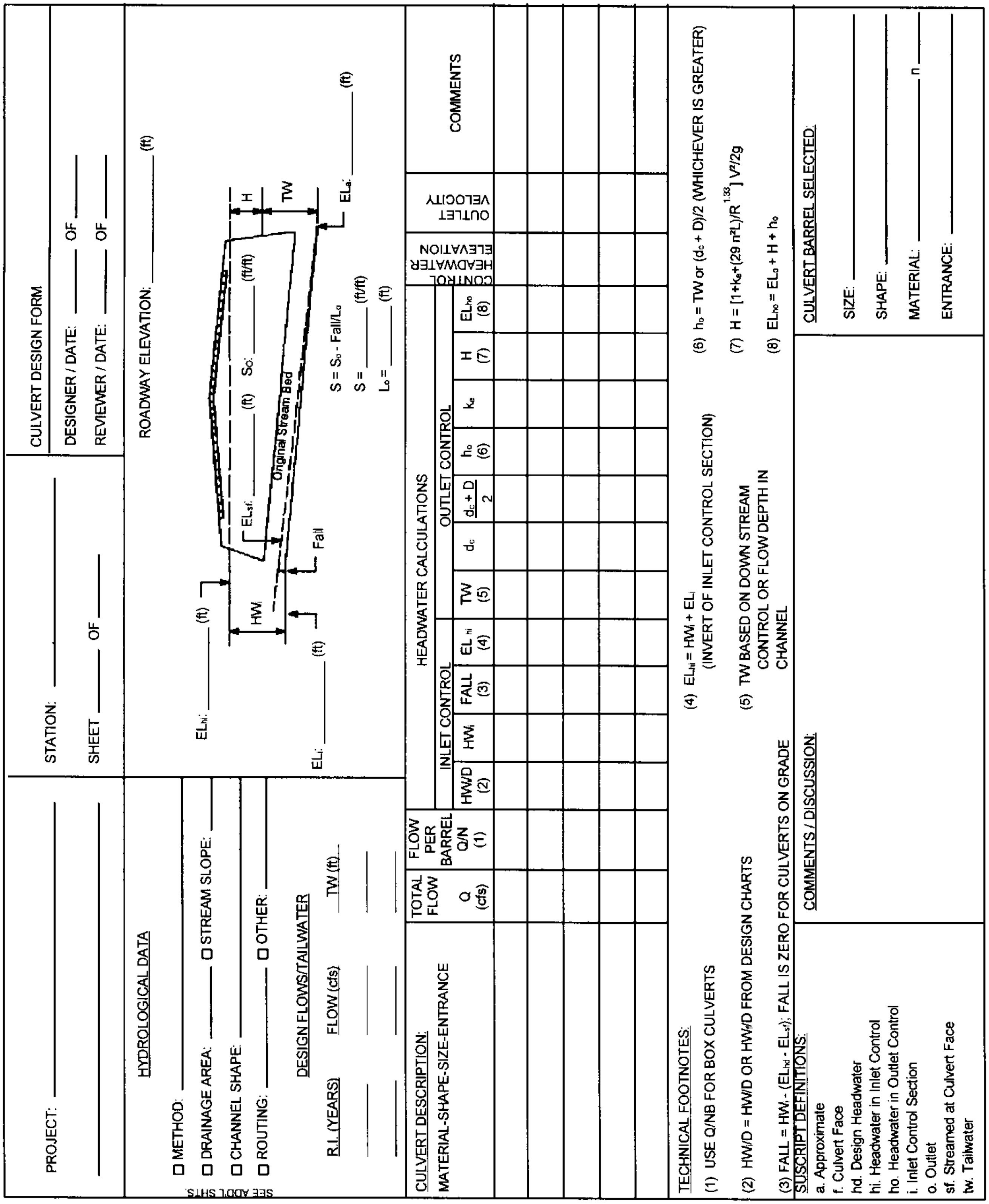
With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See *Section 4.0*.

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 flood mitigation storm flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

Figure 3.33 provides a convenient form to organize culvert design calculations.



CULVERT DESIGN FORM

PROJECT: ______________________

STATION: ______________

SHEET ______ OF ______

DESIGNER / DATE: ______ OF ______

REVIEWER / DATE: ______ OF ______

HYDROLOGICAL DATA

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

SEE ADD'L SHTS.

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: ______ (ft)

$S = S_o - Fall/L_o$

S = ______ (ft/ft)

L_o = ______ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

(7) $H = [1+k_e+(29\ n^2L)/R^{1.33}]\ V^2/2g$

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

SUSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

Figure 3.33 Culvert Design Calculation Form
(Source: HDS No. 5, 2001)

3.3.6 Design Procedures for Beveled-Edged Inlets

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.

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	90º headwalls (same for 90 º 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

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 found in Appendix D of HDS No. 5 apply only to bevels having either a 33º angle (1.5:1) or a 45º 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.

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.

Multi-barrel Installations

For multi-barrel 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 multi-barrel 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.

Multi-barrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

Skewed Inlets

It is recommended that Chart 11 found in Appendix D of HDS No. 5 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.

3.3.7 Flood Routing and Culvert Design

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.

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.

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.

3.4 Bridge Design

3.4.1 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.

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.

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.

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.

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 flood mitigation storm 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.

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.

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.

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.

3.4.2 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 in Table 3.1. There are other definitions unique to bridges which are presented here. They are defined in the TxDOT Hydraulic Design Manual.

Flow Zones and Energy Losses

Figure 3.34 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 3.35.

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.

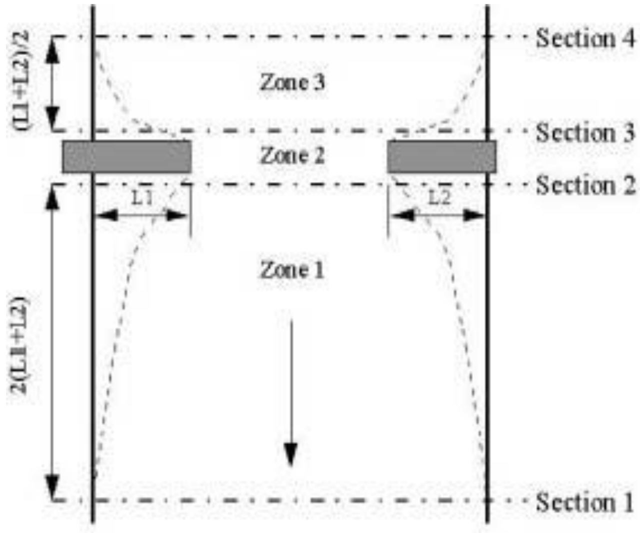


Figure 3.34 Flow Zones at Bridges
(TxDOT Hydraulic Design Manual)

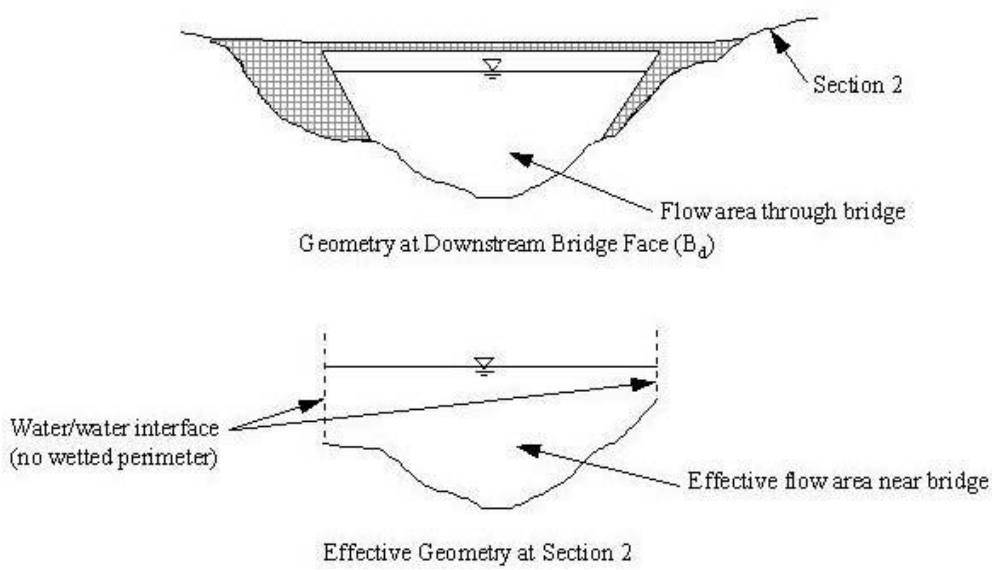


Figure 3.35 Effective Geometry for Bridge (Section 2 shown, Section 3 similar)
(TxDOT Hydraulic Design Manual)

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.

Table 3.15 Low Flow Classes

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.

3.4.3 Design Recommendations

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 recommendations should be considered for all bridge designs as applicable. See the design criteria of the local jurisdiction for specific requirements.

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 flood mitigation storm return period.

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 flood mitigation storm fully-developed discharge to check the bridge design. The flood mitigation storm discharge, assuming blockage of outlet works, with 6” of freeboard. Some municipalities may specify different design storms and freeboard requirements.

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 3.16 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 3.16 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

3.4.4 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 (Municipality, 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.

3.5 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.

American Society of Civil Engineers, 1993, Design and Construction of Urban Stormwater Management Systems, Manual and Report No. 77.

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.

Debo, Thomas N. and Andrew J. Reese, 1995. Municipal Stormwater Management. Lewis Publishers.

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

French, R. H., 1985. Open Channel Hydraulics. McGraw-Hill Book Co. New York, New York.

Federal Highway Administration, 1978. Hydraulics of Bridge Waterways. Hydraulic Design Series No. 1.

Federal Highway Administration, 1989. Bridge Waterways Analysis Model (WSPRO), Users Manual, FHWA IP-89-027.

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.

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.

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/.)

Maynord, S. T., 1987. Stable Riprap Size for Open Channel Flows. Ph.D. Dissertation. Colorado State University, Fort Collins, CO.

Larry W. Mays, Editor, 2001, Stormwater Collection Systems Design Handbook, McGraw-Hill Companies, Inc., New York, NY.

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.

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

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

Texas Department of Transportation, March 2004. Hydraulic Design Manual. Available at http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd/@Generic__BookView.

The Dewberry Companies, 2002, 2nd Edition, Land Development Handbook, McGraw-Hill Companies, Inc., New York, NY.

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 Interior, 1983. Design of Small Canal Structures.

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

4.0 Energy Dissipation

4.1 Overview

4.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater 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 stormwater 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 stormwater conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

4.1.2 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 stormwater 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.

4.1.3 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.

4.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 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 4.1 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

4.3 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 4.1 provides the riprap size recommended for use downstream of energy dissipators.

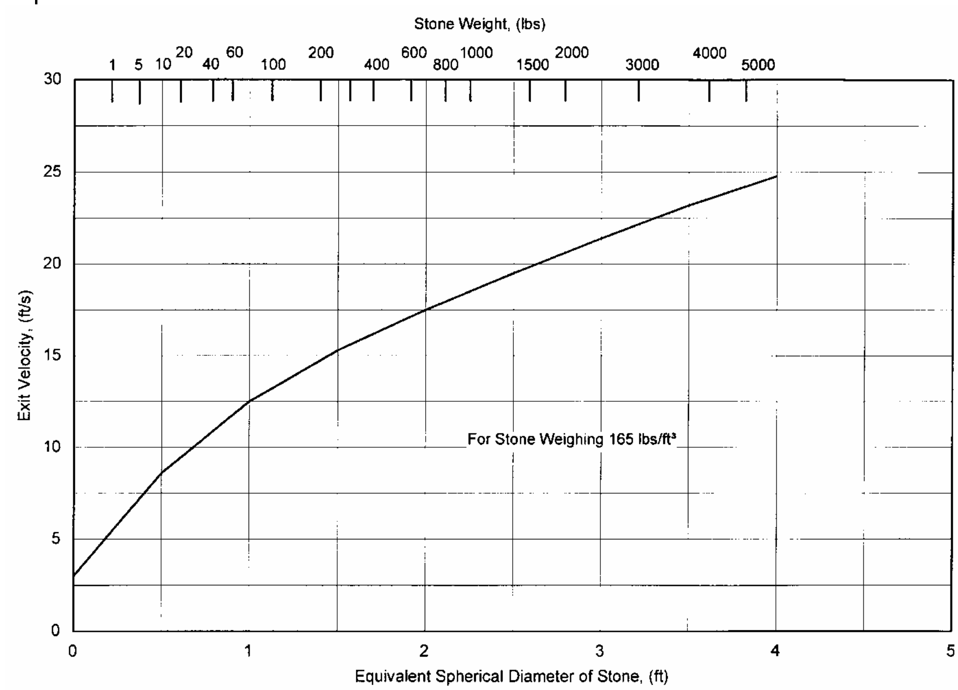


Figure 4.1 Riprap Size for Use Downstream of Energy Dissipator

(Source: Searcy, 1967)

4.4 Riprap Aprons

4.4.1 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.

4.4.2 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 4.2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 4.3 should be used.

Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 4.2 and 4.3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 4.4.

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 4.4. This will provide protection under either of the tailwater conditions.

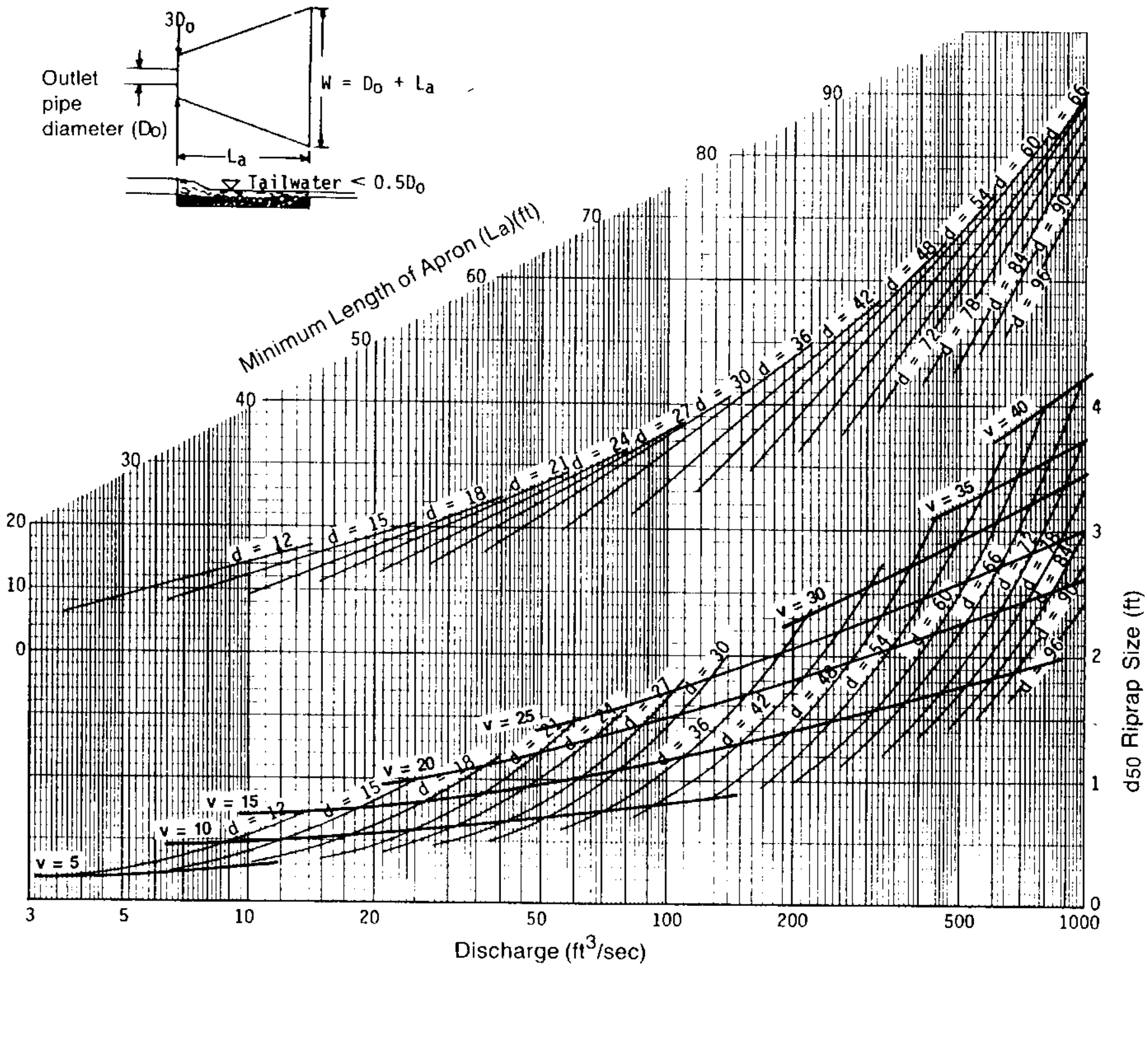
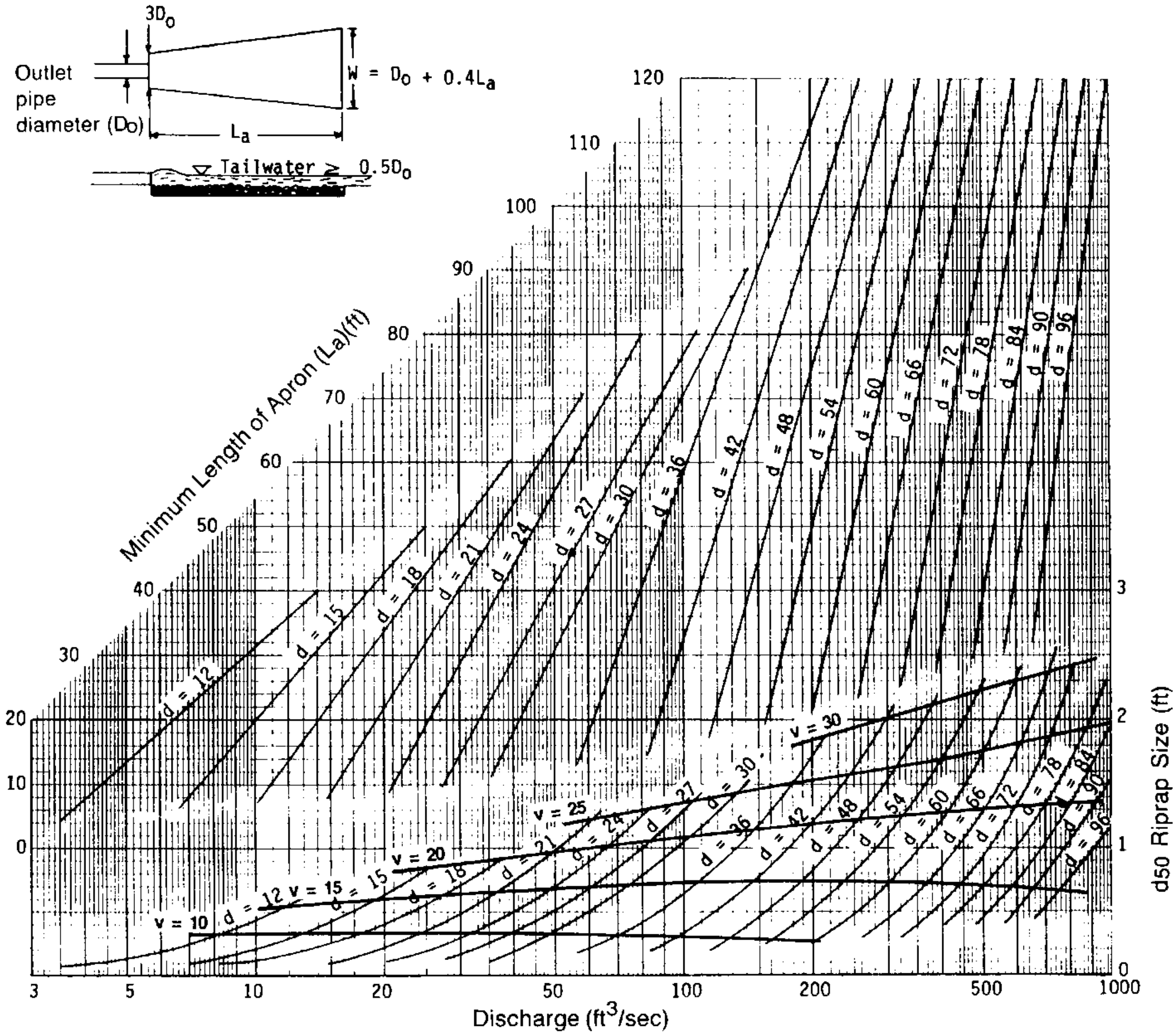


Figure 4.2 Design of Riprap Apron under Minimum Tailwater Conditions

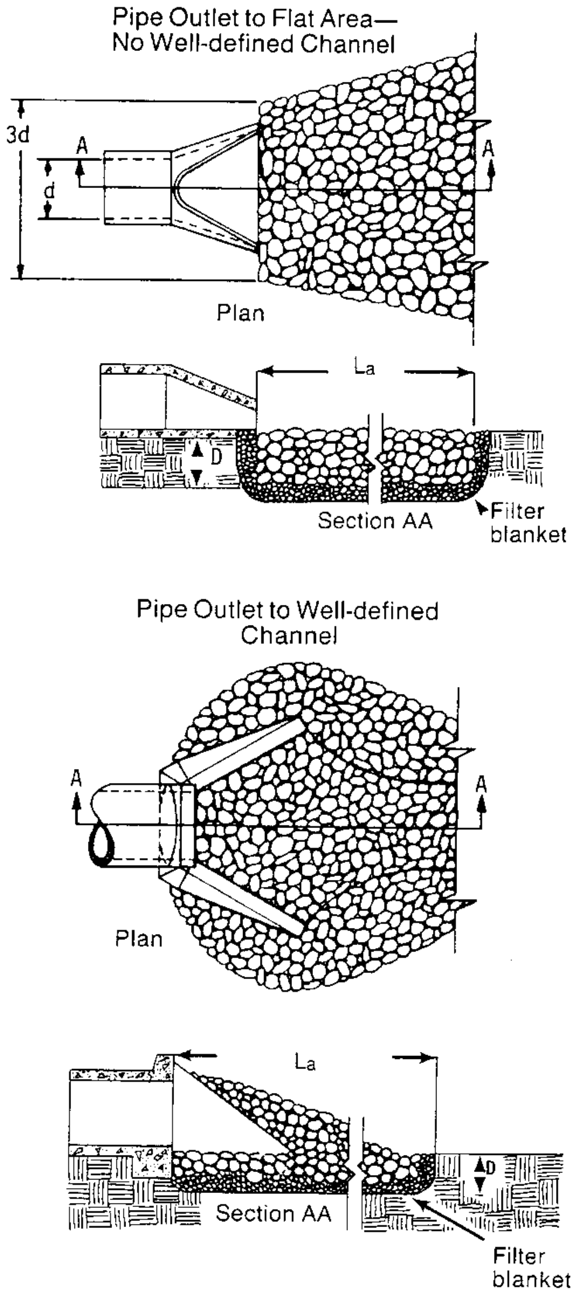
(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.3 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 4.4 Riprap Apron

(Source: Manual for Erosion and Sediment Control in Georgia, 1996)

4.4.3 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.

4.4.4 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 4.2 for minimum tailwater conditions.

By Figure 4.2, 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.

<u>Example 2</u> 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 4.3 for maximum tailwater conditions.

$v = Q/A = 600/[(5)(10)] = 12$ ft/s

On Figure 4.3, 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.

4.5 Riprap Basins

4.5.1 Description

Another method to reduce the exit velocities from stormwater 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.

4.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.5. 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.

4.5.3 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 4.6 or Figure 4.7 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 4.8, 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 4.5.

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 4.5, 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 4.9.
- Shape downstream channel and size riprap using Figure 4.1 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.

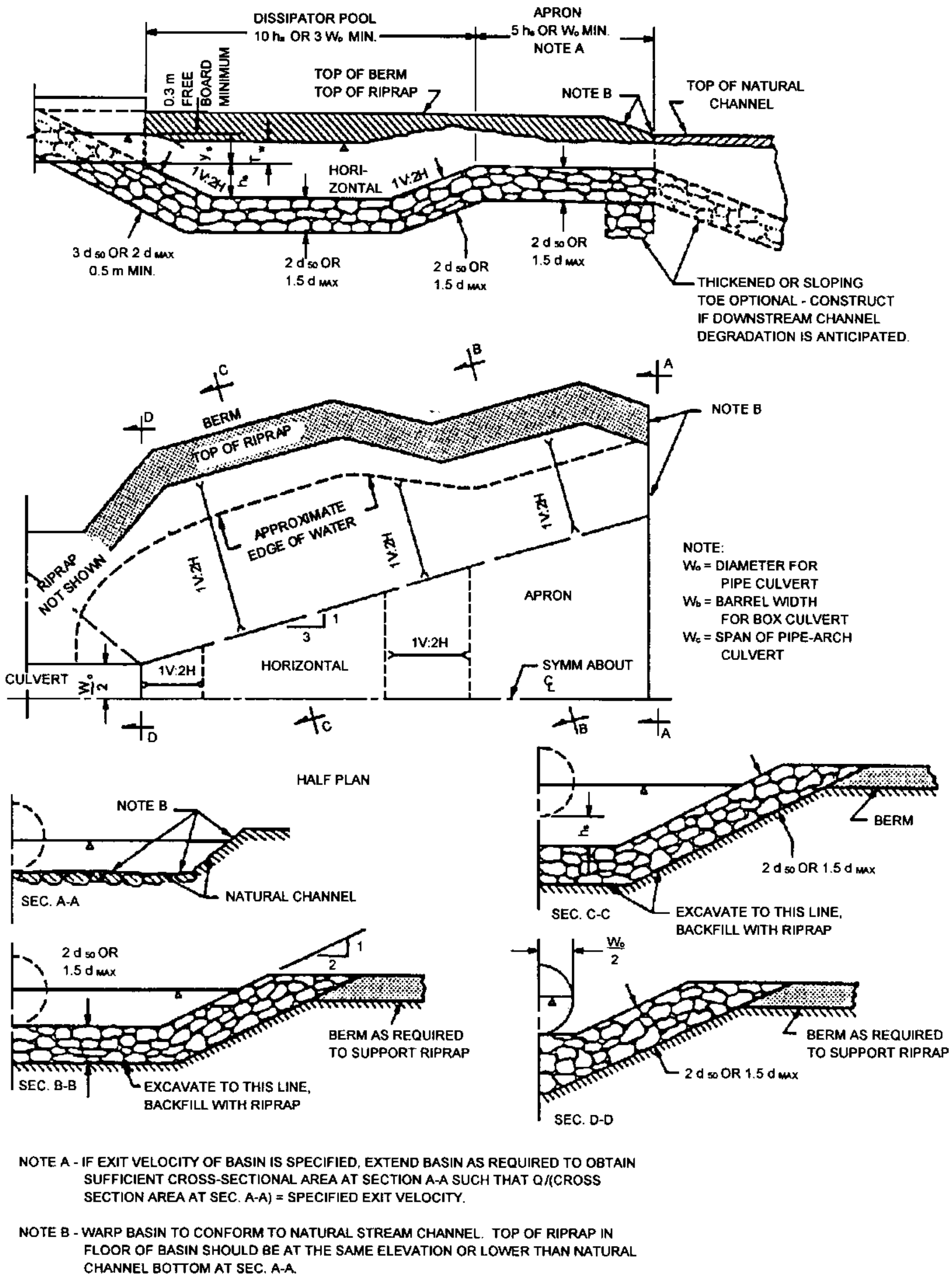


Figure 4.5 Details of Riprap Outlet Basin

(Source: HEC-14, 1983)

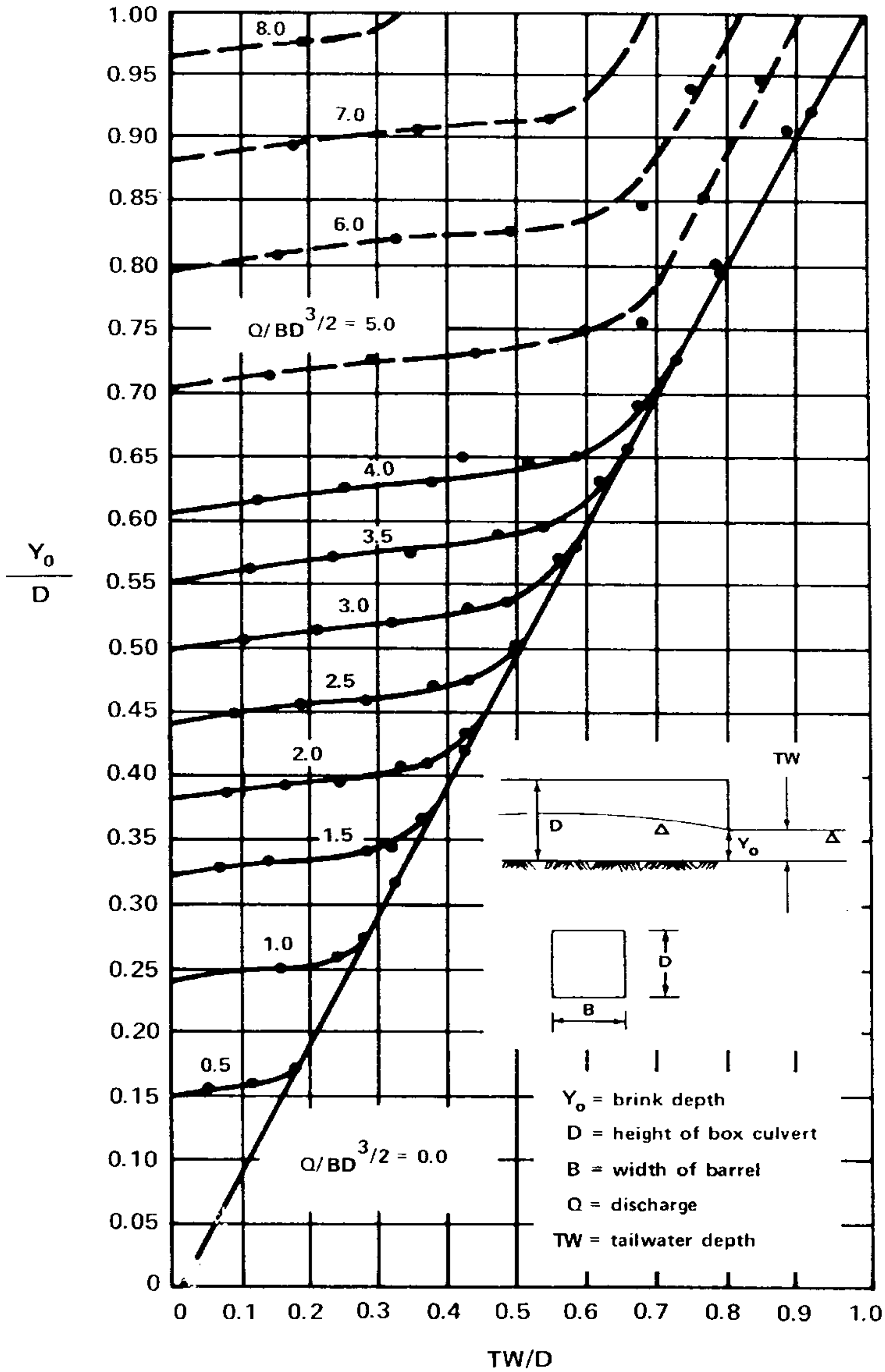


Figure 4.6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

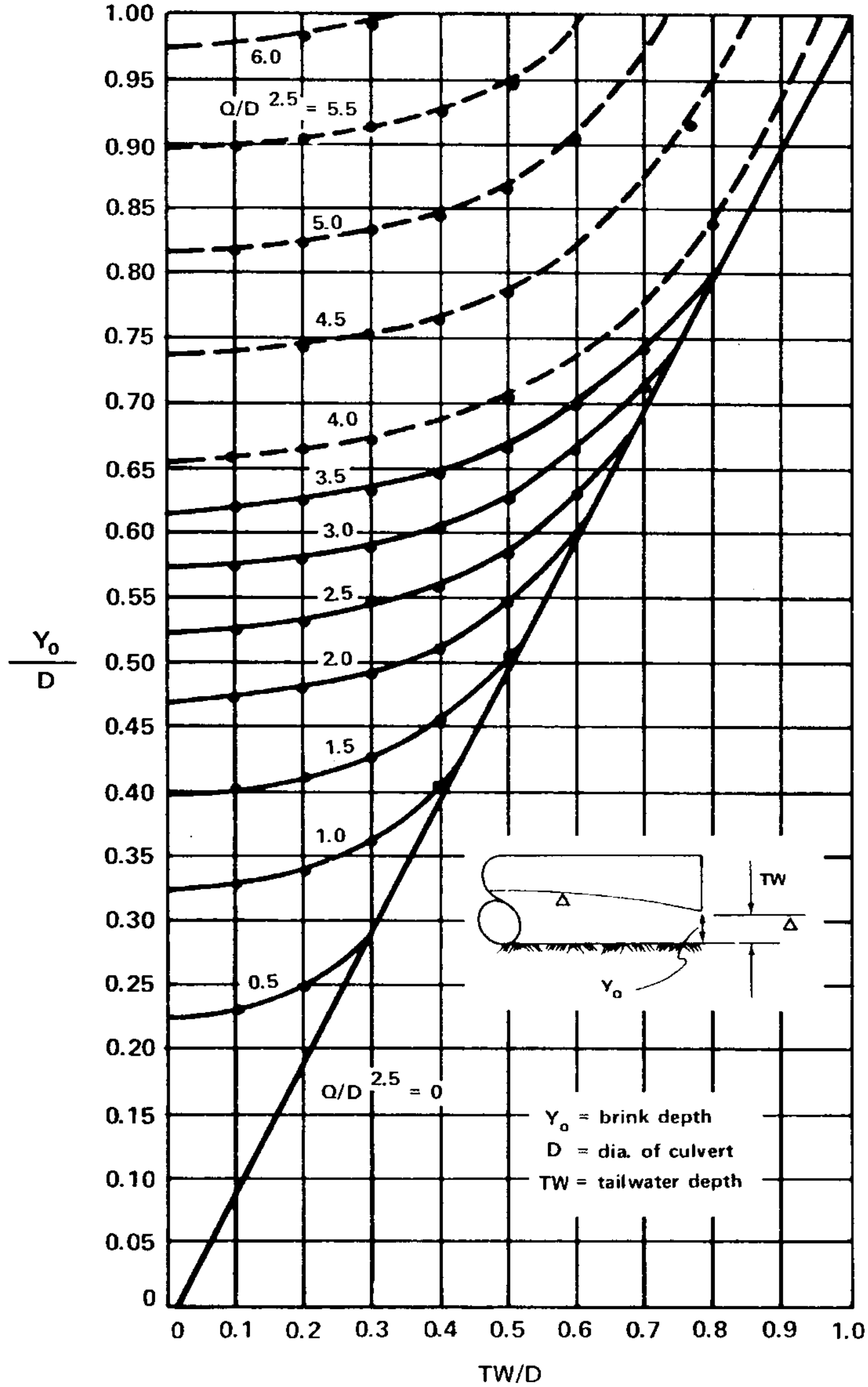


Figure 4.7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

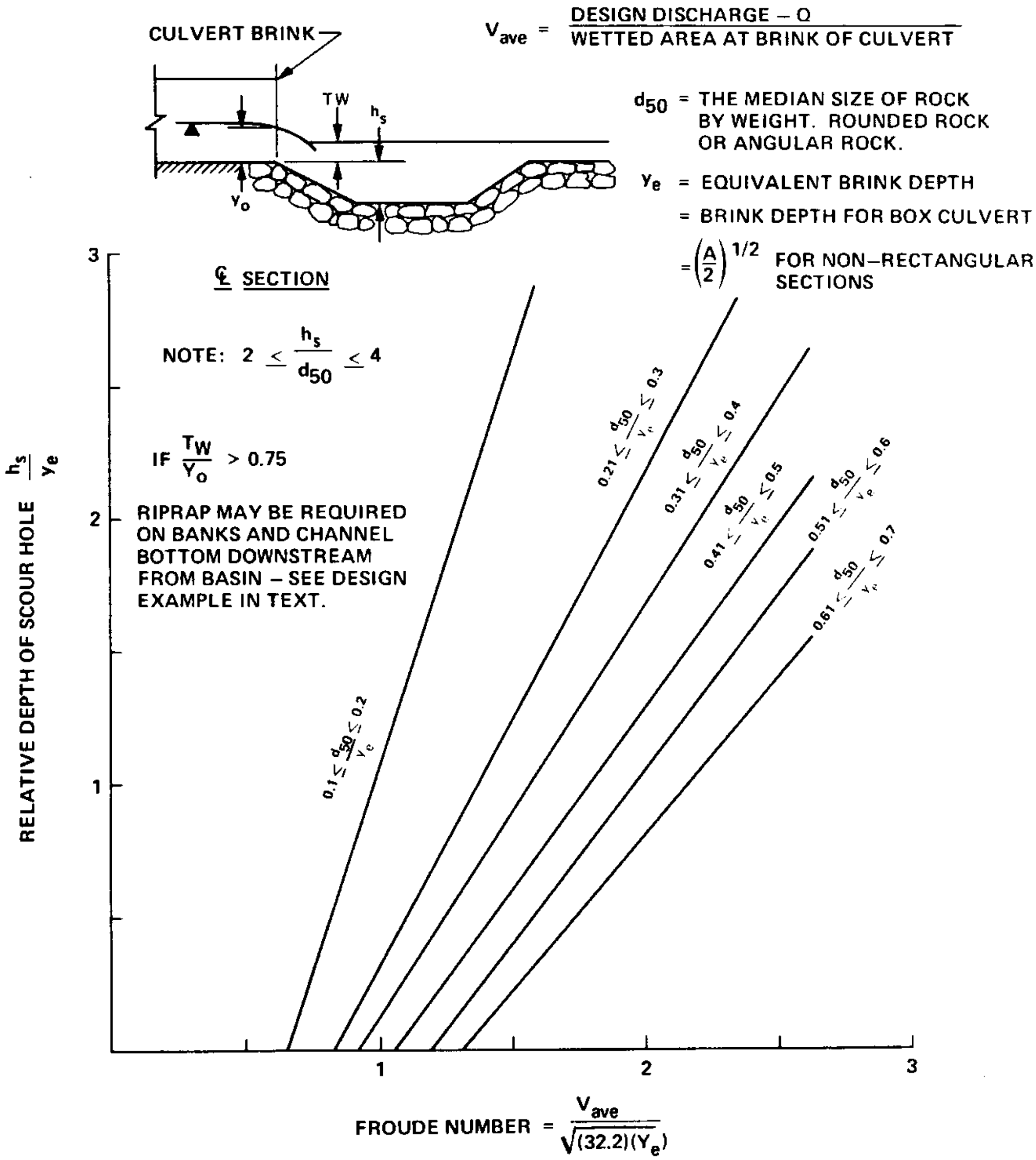


Figure 4.8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

(Source: USDOT, FHWA, HEC-14, 1983)

4.5.4 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 3.2.*

4.5.5 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

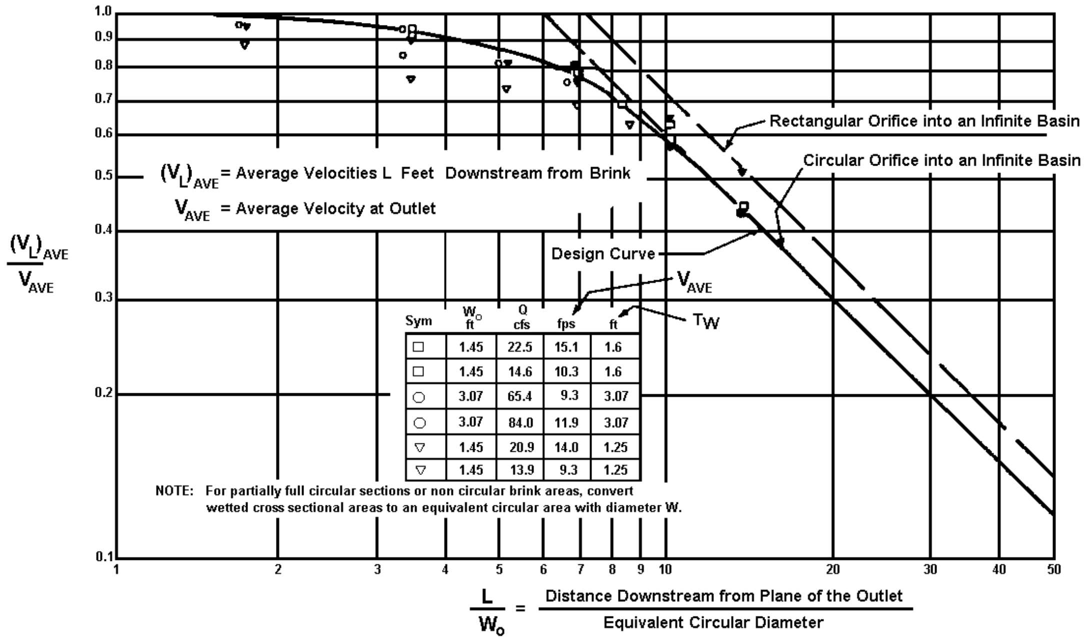
Solution: Definition of terms in Steps 1 through 5 can be found in Figures 4.5 and 4.8.

$y_o = y_e$ 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.



Sym	Wo ft	Q cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
○	3.07	65.4	9.3	3.07
○	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

Figure 4.9 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 4.8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ 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 4.5.

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 4.9 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. 9)	(Fig. 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, $y_o/D = 0.45$
$y_o = .45 \times 6 = 2.7$ 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 Table 3.7)
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 4.8, $h_s/y_e = 0.7$; therefore, $h_s = 0.7 \times 2.48 = 1.74$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From *Section 3.2.4*)
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 4.5. Figure 4.10 is provided as a convenient form to organize and present the results of riprap basin designs.

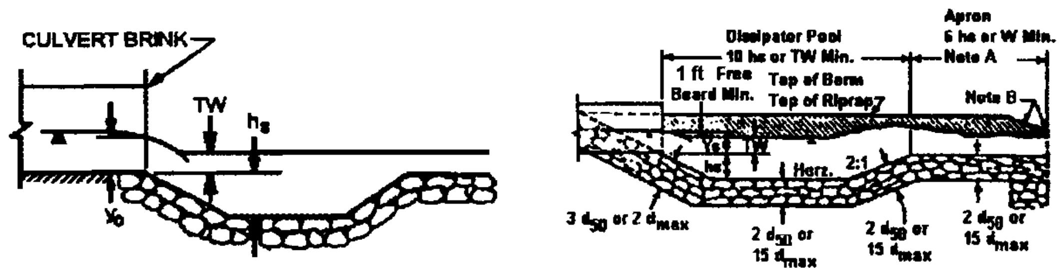
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x d_{50} thickness of riprap on the approach and the 2 x d_{50} thickness 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	
$D_E = (4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP				
L/D_E	L	V_L/V_o	V_L	D_{50}

Figure 4.10 Riprap Basin Design Form

(Source: USDOT, FHWA, HEC-14, 1983)

4.6 Baffled Outlets

4.6.1 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 4.11. 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.

4.6.2 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 4.11 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.

$$W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566} \qquad \textbf{(4.1)}$$

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 4.11, 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.

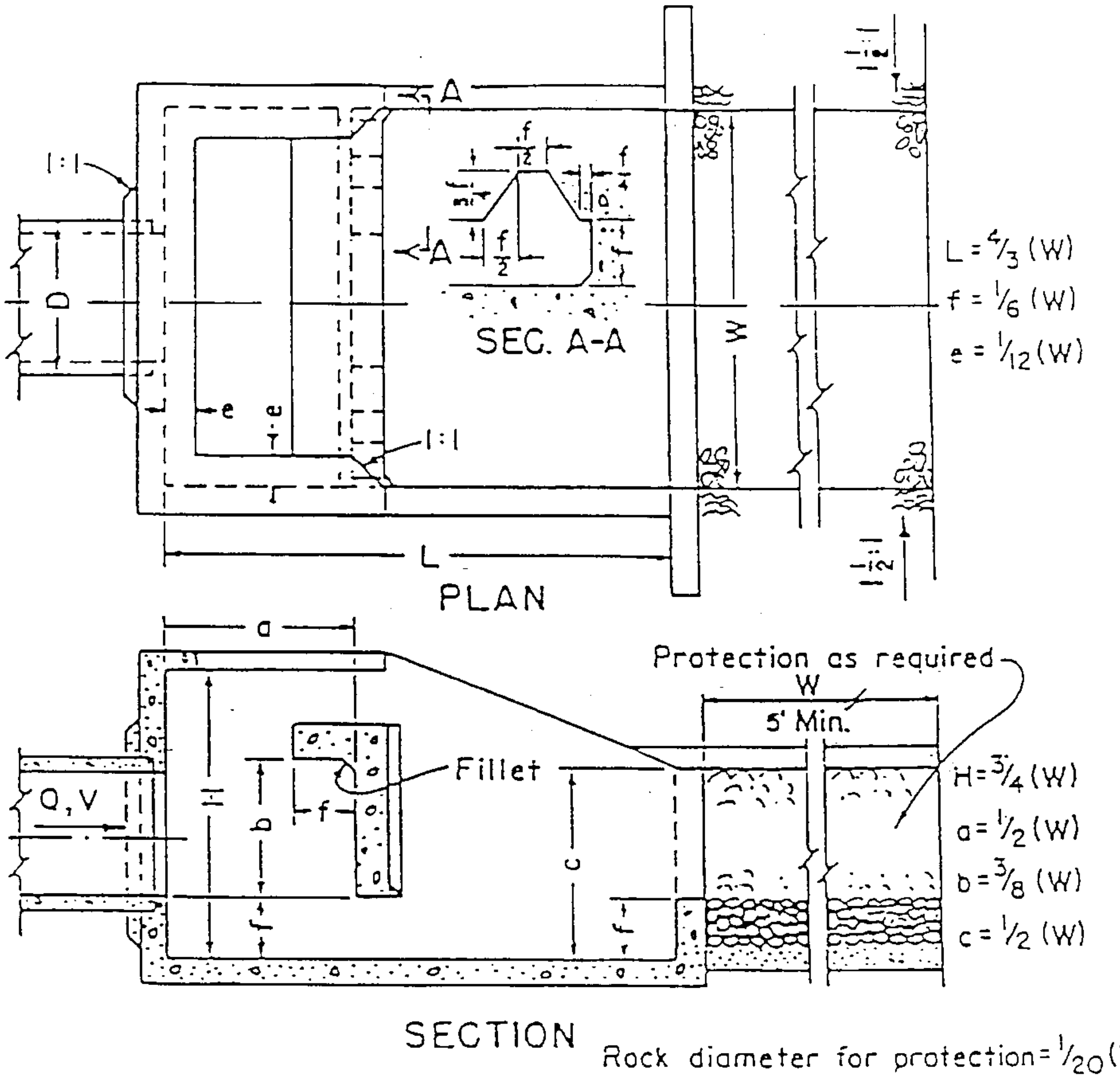


Figure 4.11 Schematic of Baffled Outlet

(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.

4.6.3 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 4.1 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 4.11):
   - 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 4.11.

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 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.

Site Development Controls:

Table of Contents

List of Tables

List of Figures

1.0 Overview of Stormwater Controls for Site Development

1.1 Stormwater Controls - Categories and Applicability

1.1.1 Introduction

Structural stormwater controls are engineered facilities intended to treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization. This section provides an overview of structural stormwater controls that can be used to address the minimum stormwater management standards outlined in *Section 1.1.2*.

In terms of the *Integrated* Design Focus Areas, a structural stormwater control, or set of structural controls, must:

- **Water Quality:** Remove pollutants in stormwater 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.

1.1.2 Control Categories

The stormwater control practices recommended in this Manual vary in their applicability and ability to meet stormwater management goals:

Primary Controls

Primary Structural Stormwater Controls have the ability to fully address one or more of the Steps in the *integrated* Design Focus Areas 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 stormwater management facilities for a site wherever feasible and practical.

Secondary Controls

However, a number of structural controls are recommended only 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 Focus Areas, (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 1.1 lists the structural stormwater 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 Sections 2 through 28 of this manual.

<table>
<tr><th colspan="2">Table 1.1 Structural Controls</th></tr>
<tr><th>Structural Control</th><th>Description</th></tr>
<tr><td>Bioretention Areas</td><td>Bioretention areas are shallow stormwater basins or landscaped areas which utilize engineered soils and vegetation to capture and treat stormwater runoff. Runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil.</td></tr>
<tr><td>Channels
• Enhanced Swale (Dry, Wet, or Wetland)
• Grass Channel (biofilter)</td><td>• Enhanced swales are vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means
• Grass channels provide “biofiltering” of stormwater 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.</td></tr>
<tr><td>Chemical Treatment
• Alum Treatment</td><td>Alum treatment provides for the removal of suspended solids from stormwater 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.</td></tr>
<tr><td>Conveyance Components
• Culvert
• Inlet
• Pipe Systems
• Energy Dissipators
• Open Conveyance Channel</td><td>• A culvert is a short, closed (covered) conduit that conveys stormwater 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 stormwater controls and receiving waters.
Culverts, inlets, and pipe systems alone do not provide water quality treatment.</td></tr>
<tr><td>Detention
• Dry Detention / Dry Extended Detention Basins
• Multi-Purpose Detention Areas
• Underground Detention</td><td>• Dry detention basins and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater 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.</td></tr>
</table>

Table 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 stormwater 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 stormwater 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 stormwater 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 stormwater 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 stormwater 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 stormwater devices, such as draining downspouts or planter boxes.

Table 1.1 Structural Controls

Structural Control	Description
Stormwater Ponds • Micropool Extended Detention Pond • Multiple Pond Systems • Wet Extended Detention Pond • Wet Pond	*Stormwater ponds* are constructed stormwater 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 Stormwater Controls	*Proprietary controls* are manufactured structural control systems available from commercial vendors designed to treat stormwater 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.
Stormwater Wetlands • Extended Detention Shallow Wetland • Pocket Wetland • Pond/Wetland Systems • Shallow Wetland • Submerged Gravel Wetlands	• *Stormwater wetlands* are constructed wetland systems used for stormwater management. Stormwater 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 stormwater 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.

1.1.3 Using Other or New Structural Stormwater 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 stormwater 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.

1.2 Suitability of Stormwater Controls

Some structural stormwater controls are intended to provide water quality treatment for stormwater runoff. Though most of these structural controls provides pollutant removal capabilities, the relative capabilities vary between structural control practices and for different pollutant types.

1.2.1 Water Quality

Pollutant removal capabilities for a given structural stormwater 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 1.2 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 stormwater 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 stormwater controls in series is provided in *Section 1.6*.

For additional information and data on the range of pollutant removal capabilities for various structural stormwater controls, the reader is referred to the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

Table 1.2 Design Pollutant Removal Efficiencies for Stormwater 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
Filter Strip	50	20	20	---	40
Dry Detention	65	50	30	70	---
Organic Filter	80	60	40	50	75

Table 1.2 Design Pollutant Removal Efficiencies for Stormwater Controls (Percentage)

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metals
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
Stormwater 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	---	---	---	---	---
Stormwater 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

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, stormwater ponds, stormwater wetlands, and pipe systems.

1.2.3 Flood Control

- **On-Site:** These controls have the ability to safely convey stormwater 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, stormwater ponds, conveyance components (inlets and pipe systems), and stormwater 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, stormwater ponds, and stormwater wetlands.

1.3 Stormwater Control Selection

1.3.1 Control Screening Process

Outlined below is a screening process for structural stormwater 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:

- Stormwater 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 stormwater 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 stormwater 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 1.3, 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>Stormwater 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 Focus Areas. 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 flood mitigation 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 stormwater 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.

Step 2 Specific Criteria

The last three categories in the Structural Control Screening matrix (Table 1.3) 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 stormwater 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 stormwater controls is fundamentally influenced by the nature of the downstream water body that will be receiving the stormwater 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.

Table 1.3 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

Table 1.3 Structural Control Screening Matrix

✓ **-** 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

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

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 stormwater 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 stormwater 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 1.4) 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 stormwater control within the regulated area.
- Structural stormwater 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 stormwater structural controls. Consultation with the appropriate regulatory agency is the best strategy.

Table 1.4 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 stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater 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 stormwater 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 stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater should be treated prior to discharge into Waters of the U.S. • In-stream ponds for stormwater 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.
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 Stormwater 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.

Table 1.4 Location and Permitting Checklist

Site Feature	Location and Permitting Guidance
Stream Buffer Check with appropriate review authority whether stream buffers are required	• Consult local authority for stormwater 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 stormwater 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 stormwater infiltration. • 50-foot setback for all other structural controls.

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 stormwater 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. Stormwater runoff from the site is expected to exhibit fairly high sediment levels and seasonally high phosphorus levels (due to turf grass management).

Table 1.5 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 stormwater 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 1.5 Sample Structural Control Selection Matrix

Structural Control Alternative	Stormwater 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**				

Table 1.5 Sample Structural Control Selection Matrix						
Structural Control Alternative	**Stormwater Treatment Suitability**	**Site Applicability**	**Implementation Considerations**	**Physiographic Factors/Soils**	**Special Watershed Considerations**	**Other Issues**
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

1.4 On-Line Versus Off-Line Structural Controls

1.4.1 Introduction

Structural stormwater 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 flood mitigation storm 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 1.1 shows an example of an off-line sand filter and an on-line enhanced dry swale.

1.4.2 Flow Regulators

Flow regulation to off-line structural stormwater controls can be achieved by either:

- Diverting the water quality volume or other specific maximum flow rate to an off-line structural stormwater 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 1.4 of the Water Quality Technical Manual.*

Flow regulators can be flow splitter devices, diversion structures, or overflow structures. A number of examples are shown in Figures 1.2 through 1.4.

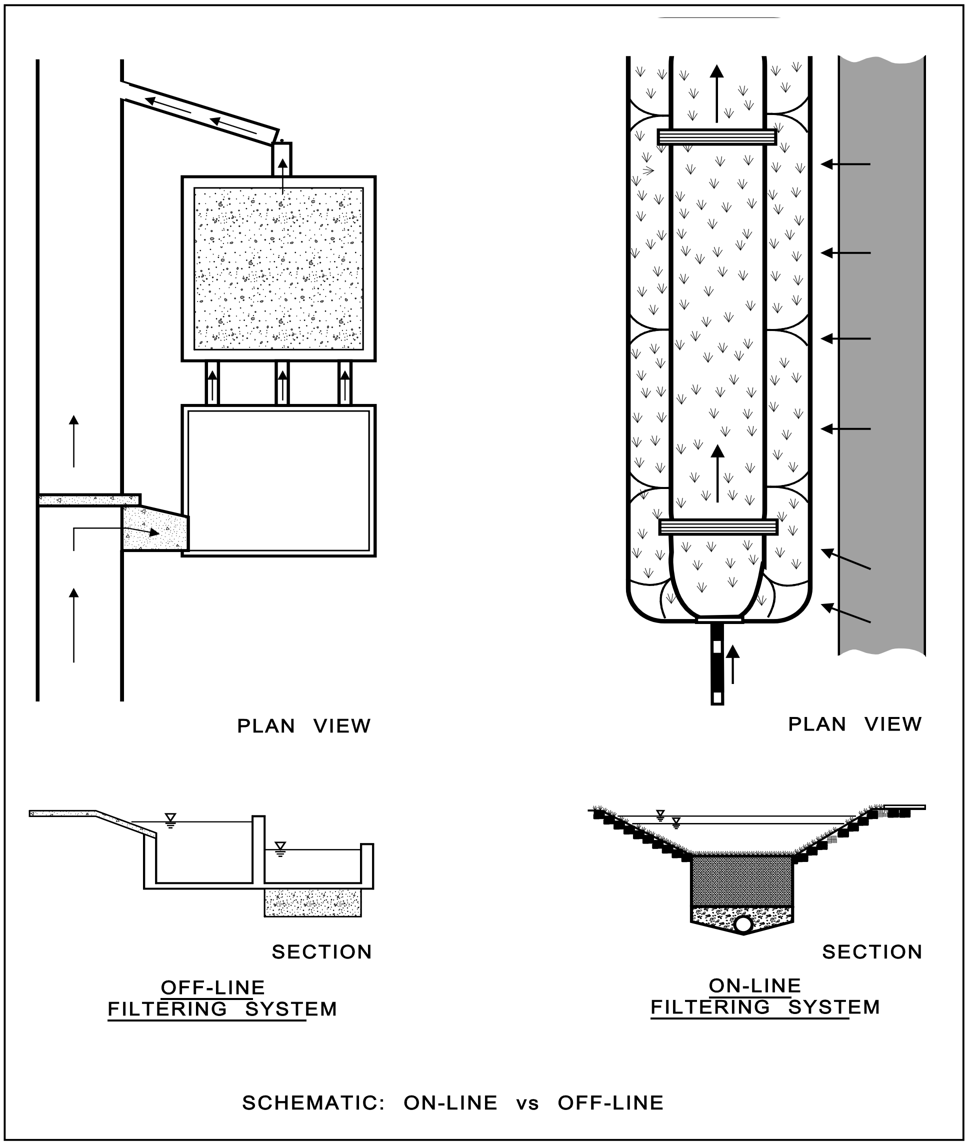


Figure 1.1 Example of On-Line versus Off-Line Structural Controls
(Source: CWP, 1996)

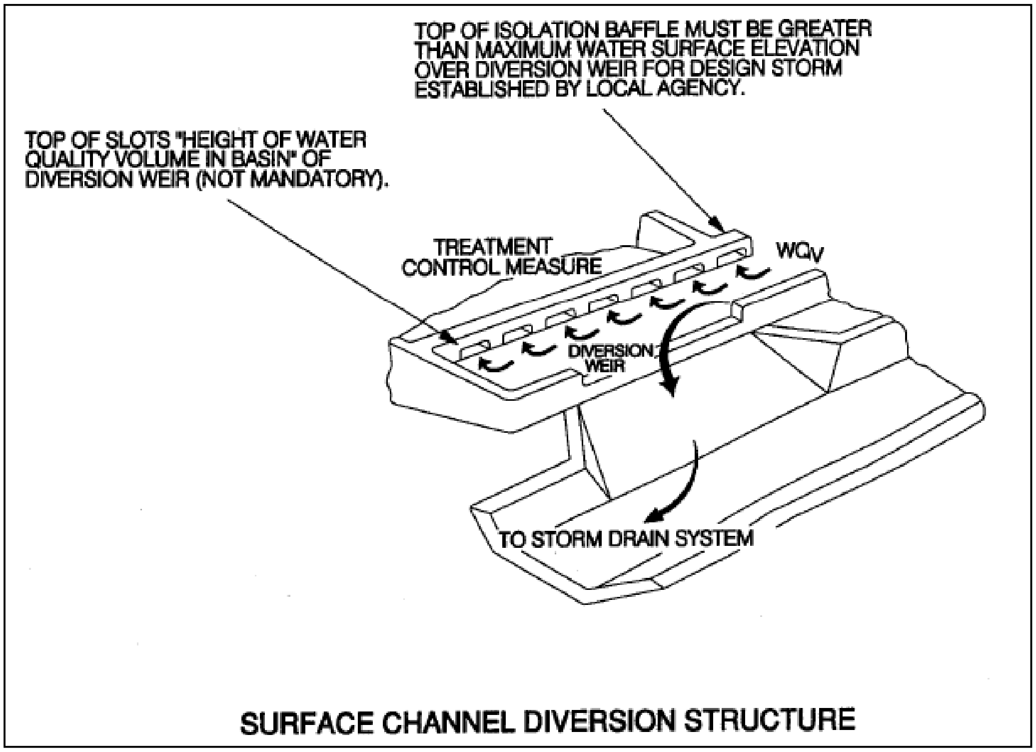


Figure 1.2 Pipe Interceptor Diversion Structure
(Source: City of Sacramento, 2000)

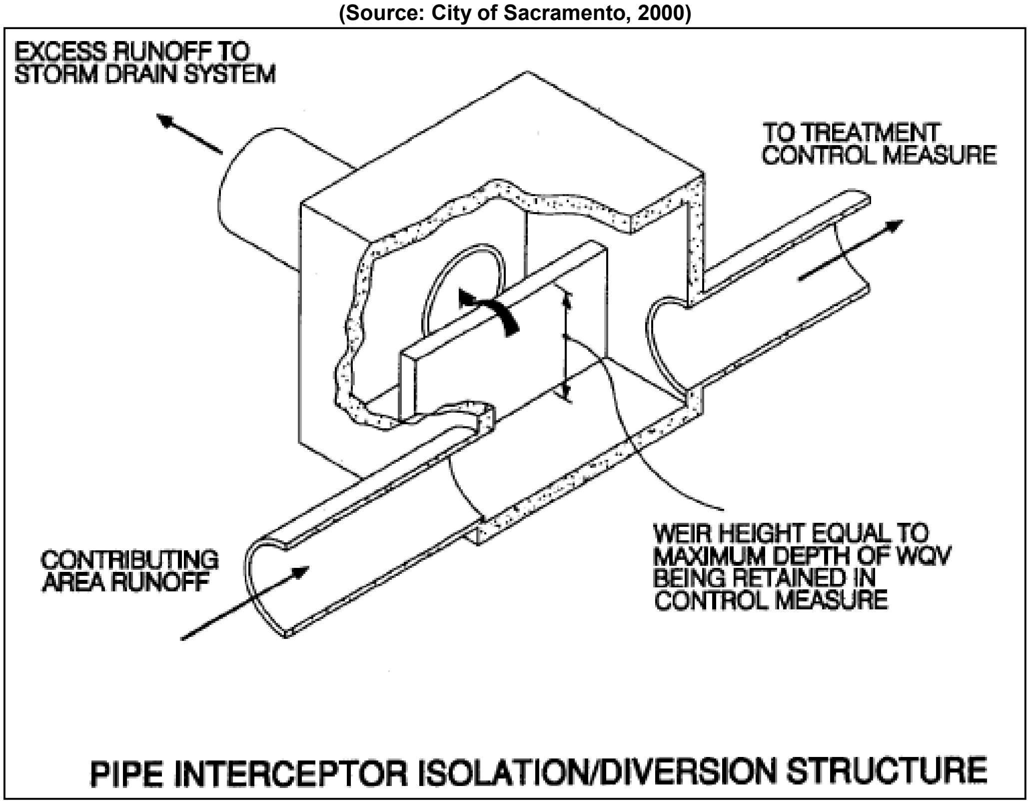


Figure 1.3 Surface Channel Diversion Structure
(Source: City of Sacramento, 2000)

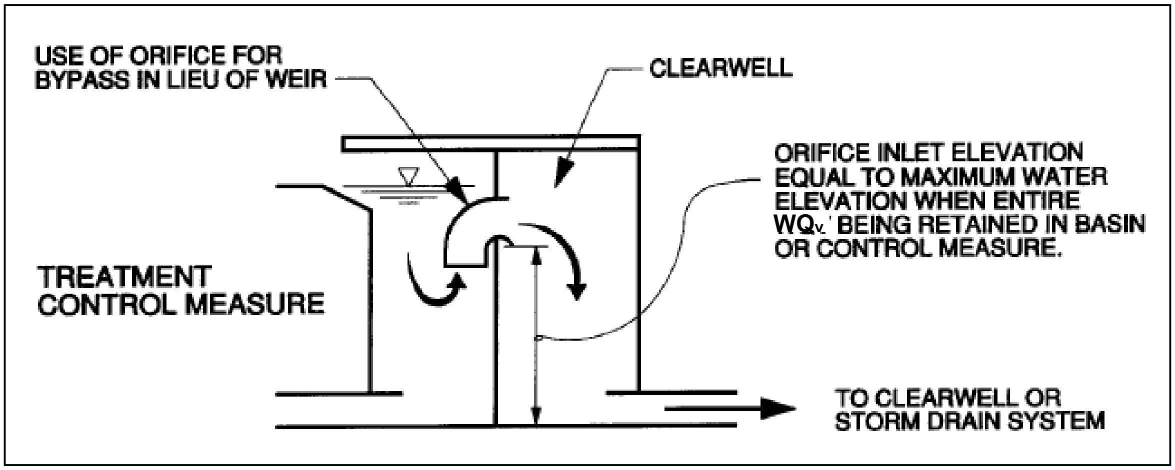


Figure 1.4 Outlet Flow Regulator
(Source: City of Sacramento, 2000)

1.5 Regional Versus On-Site Stormwater Management

1.5.1 Introduction

Using individual, on-site structural stormwater controls for each development is the typical approach for controlling stormwater 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 stormwater controls in a subwatershed rather than require on-site controls (see Figure 1.5). For this Manual, regional stormwater controls are defined as facilities designed to manage stormwater 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.

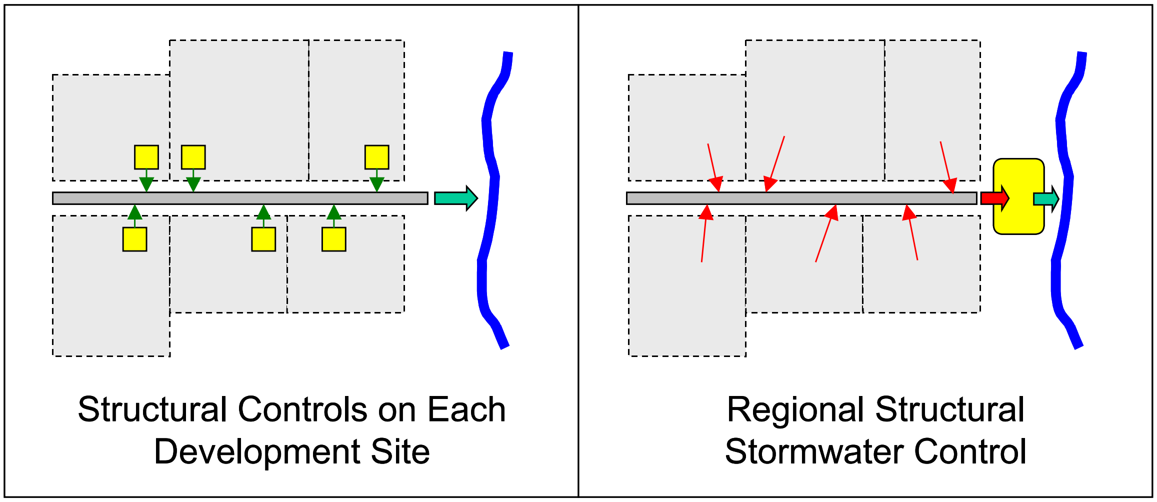


Figure 1.5 On-Site versus Regional Stormwater Management

1.5.2 Advantages and Disadvantages of Regional Stormwater Controls

Regional stormwater 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 stormwater 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 stormwater 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 stormwater utility or some other program to fund and implement stormwater 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 stormwater controls.

<u>Advantages of Regional Stormwater Controls</u>

- **Reduced Construction Costs** – Design and construction of a single regional stormwater 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 stormwater 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 stormwater 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 stormwater 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 stormwater facilities can serve as a recreational and aesthetic amenity for a community.

<u>Disadvantages of Regional Stormwater Controls</u>

- **Location and Siting** – Regional stormwater 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 stormwater facility.
- **Need for Planning** – The implementation of regional stormwater 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.

1.5.3 Important Considerations for the Use of Regional Stormwater Controls

If a community decides to implement a regional stormwater 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 stormwater discharges, and it is U.S. EPA policy to not allow regional stormwater 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 stormwater 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 stormwater master planning or watershed planning to be effective.

1.6 Using Structural Stormwater Controls in Series

1.6.1 Stormwater Treatment Trains

The minimum stormwater management standards are an integrated planning and design focus areas whose components work together to limit the adverse impacts of urban development on downstream waters and riparian areas. This approach is sometimes called a stormwater "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 1.6.

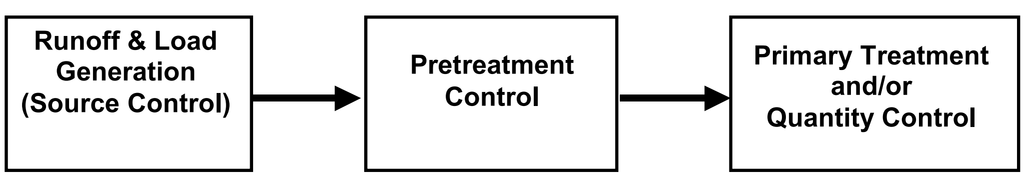


Figure 1.6 Generalized Stormwater Treatment Train

<u>Runoff and Load Generation</u> – 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 stormwater pollutants from the source.

<u>Pretreatment</u> – 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

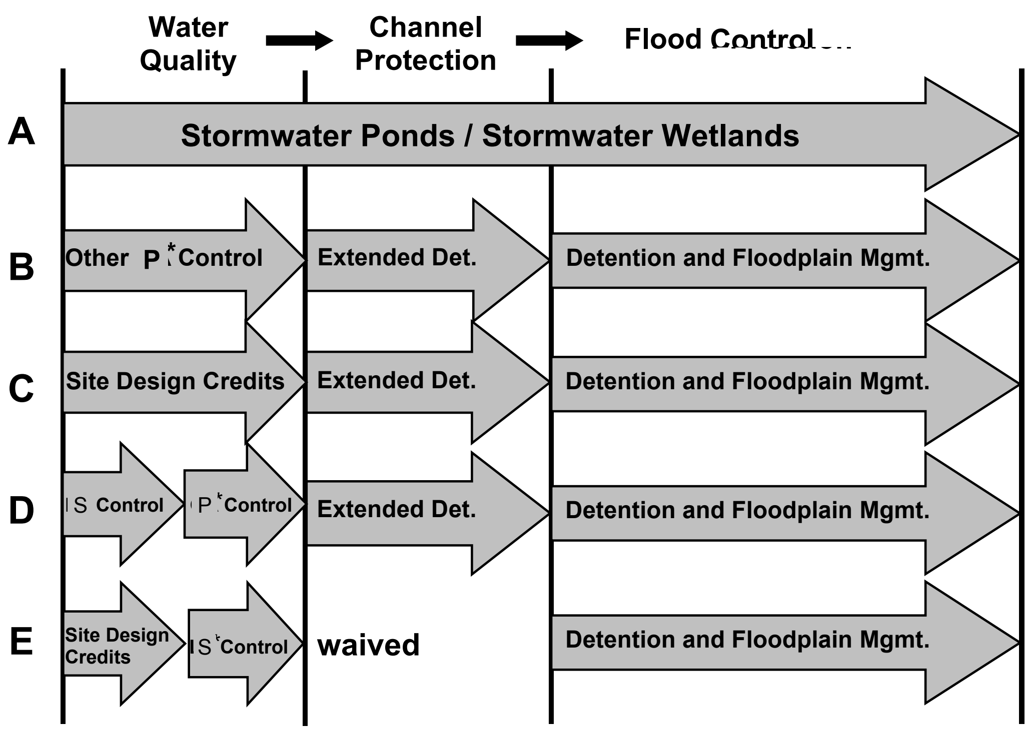
calculable water quality benefits that may be applied towards meeting the WQ_v treatment requirement. These measures include:

- The use of stormwater *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.

1.6.2 Use of Multiple Structural Controls in Series

Many combinations of structural controls in series may exist for a site. Figure 1.7 provides a number of hypothetical examples of how the *integrated* Design Focus Areas may be addressed by using structural stormwater controls.



***P - Primary Control and S - Secondary Control Limited Application.**

Figure 1.7 Examples of Structural Controls Used in Series

Referring to Figure 1.7 by line letter:

A. Two structural controls achieving Primary TSS removal each, *stormwater ponds* and *stormwater wetlands*, can be used to meet all of the requirements of the *integrated* Design Focus Areas 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 Focus Areas. 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 stormwater controls are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. Figures 1.8 through 1.10 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 1.8 rooftop runoff drains over grassed yards to backyard grass channels. Runoff from front yards and driveways reaches roadside grass channels. Finally, all stormwater flows to a extended detention micropool stormwater pond.

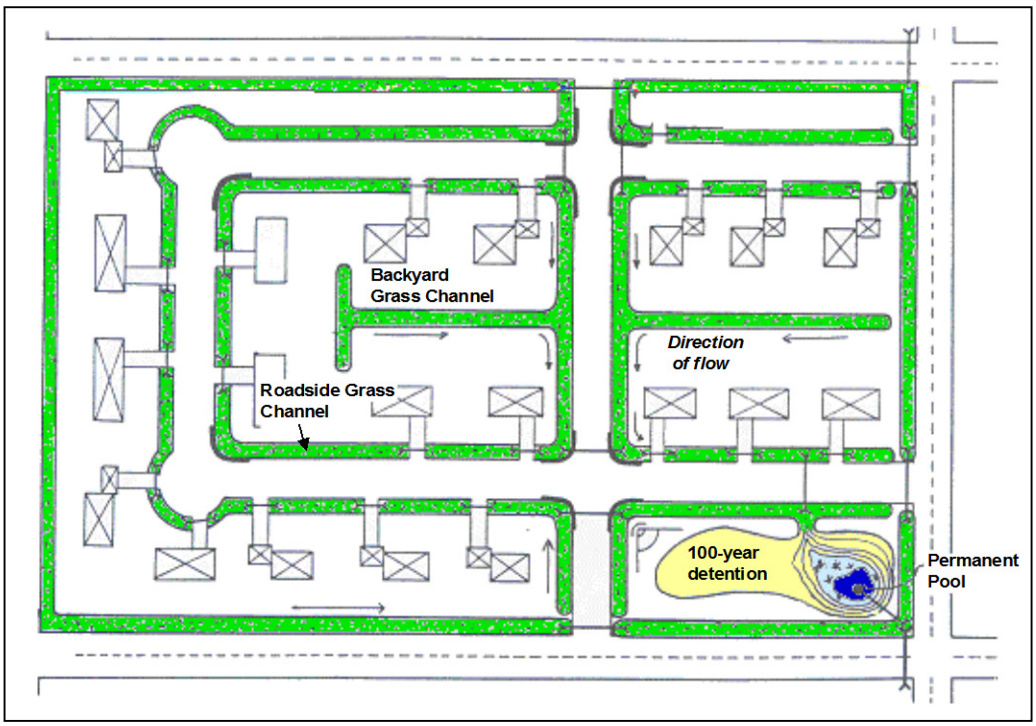


Figure 1.8 Example Treatment Train – Residential Subdivision
(Adapted from: NIPC, 2000)

A gas station and convenience store is depicted in Figure 1.9. 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 stormwater control for streambank protection is required as the system drains to the municipal storm drain pipe system. Flood control is provided by a regional stormwater control downstream.

Figure 1.10 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.

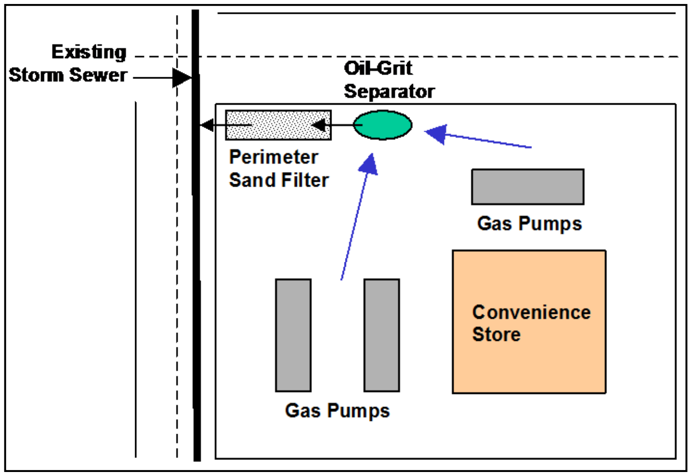


Figure 1.9 Example Treatment Train – Small Commercial Site

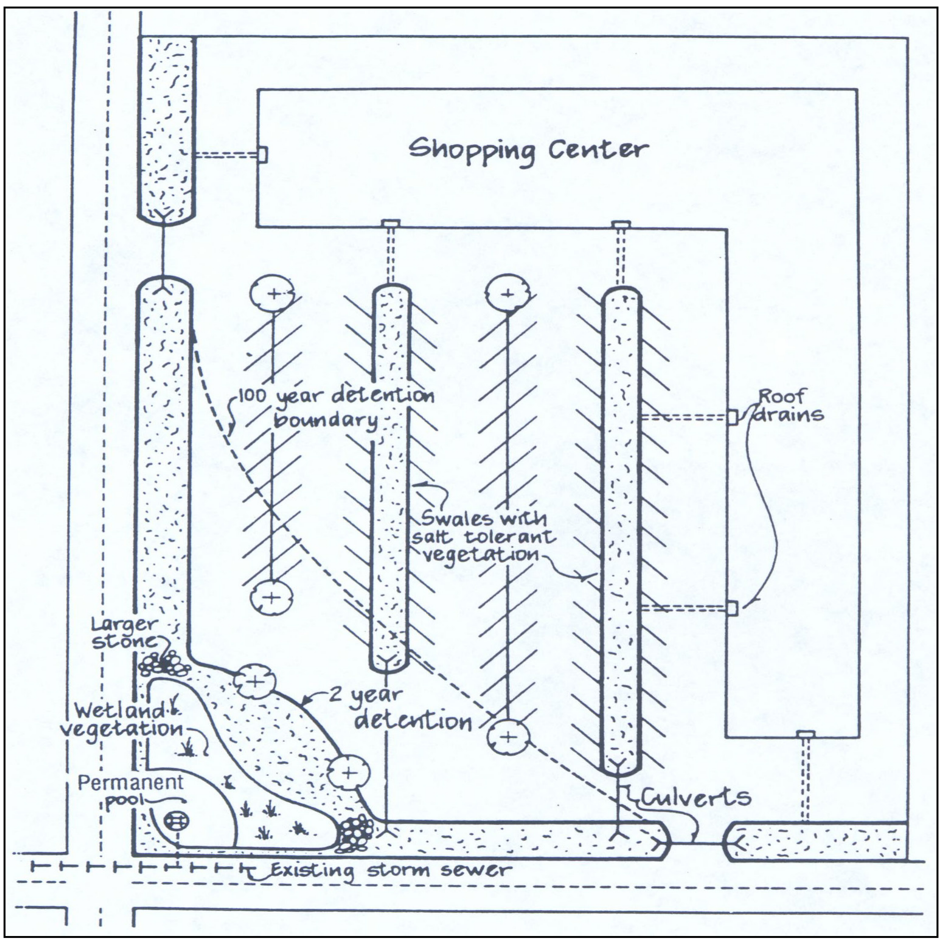


Figure 1.10 Example Treatment Train – Large Commercial Development
(Source: NIPC, 2000)

1.6.3 Calculation of Pollutant Removal for Structural Controls in Series

For two or more structural stormwater 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 stormwater pond facility will have a much higher pollutant removal percentage for very turbid runoff than for clearer water. When two stormwater 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 equations are used to determine the pollutant removal:

For treatment trains with two BMPs the following equation is used.

$$\boldsymbol{E = A + B - \{(A * B)/100\}}$$

where:

E = total efficiency
A = efficiency of first or upstream BMP
B = efficiency of second BMP

For treatment trains with three BMPs the following equation is used.

$$\boldsymbol{E = 0.95 * [A_B + C - \{(A_B * C)/100\}]}$$

where:

E = total efficiency
A_B = A+B-{(A*B)/100}
A = efficiency of first or upstream BMP
B = efficiency of second BMP
C = efficiency of third or downstream BMP

Example

TSS is the pollutant of concern and a commercial device is inserted that has a 20% sediment removal rate. A stormwater pond is designed at the site outlet. A second stormwater 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 (Stormwater Pond 1) = 80% TSS removal

Control 3 (Stormwater Pond 2) = 80% TSS removal

Apply the equations to calculate the total efficiency:

$E = 0.95*[A_B+C - \{(A_B*C)/100\}]$

where $A_B = A+B - \{(A*B)/100\} = 20+80 – \{(20*80)/100\} = 84$

$E = 0.95*[84+80 - \{(84*80)/100\}] = 91.96$

For the treatment train in total = **91.96% removal**

1.6.4 Routing with WQv 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 1.11 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.

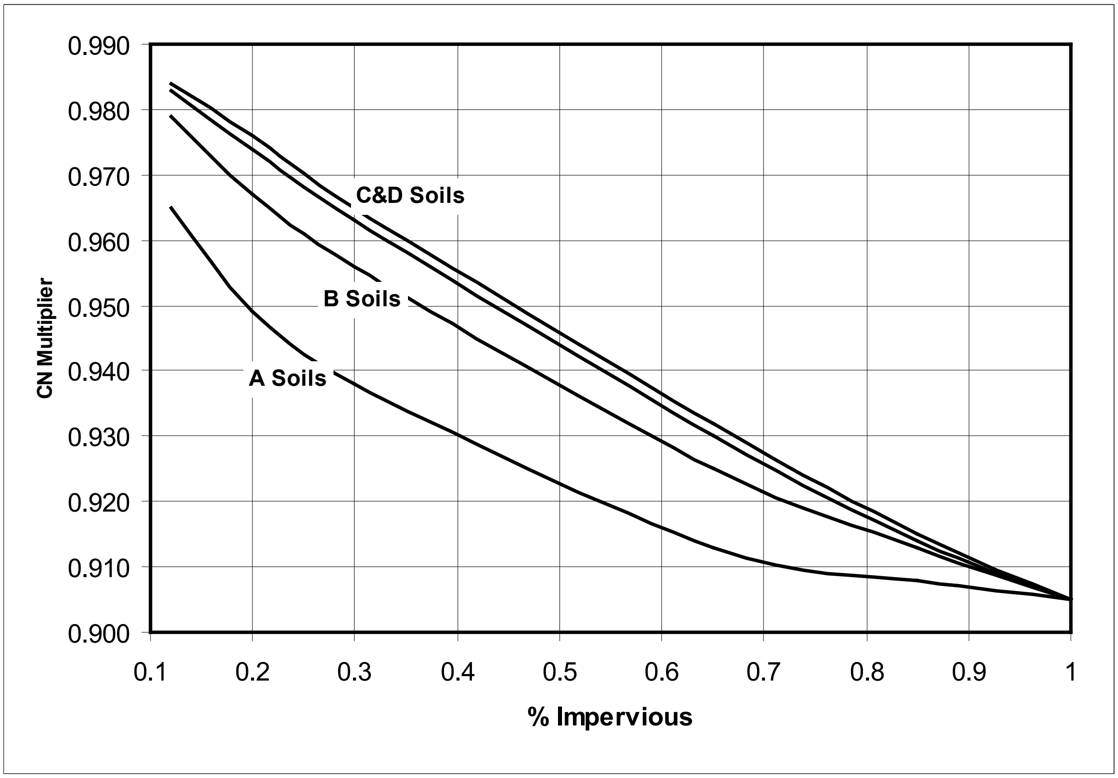


Figure 1.11 Curve Number Adjustment Factor

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 1.11 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.

2.0 Bioretention

Structural Stormwater Control

Description: Shallow stormwater 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

STORMWATER 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

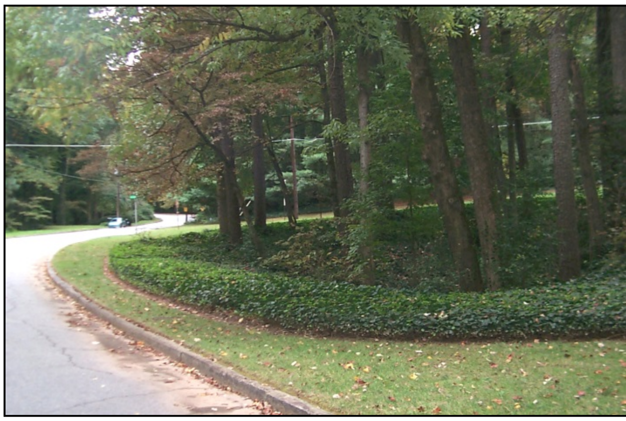
L=Low **M**=Moderate **H**=High

2.1 General Description

Bioretention areas (also referred to as *bioretention filters* or *rain gardens*) are structural stormwater 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 stormwater 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 2.1 and 2.2 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 2.1 Bioretention Area Examples

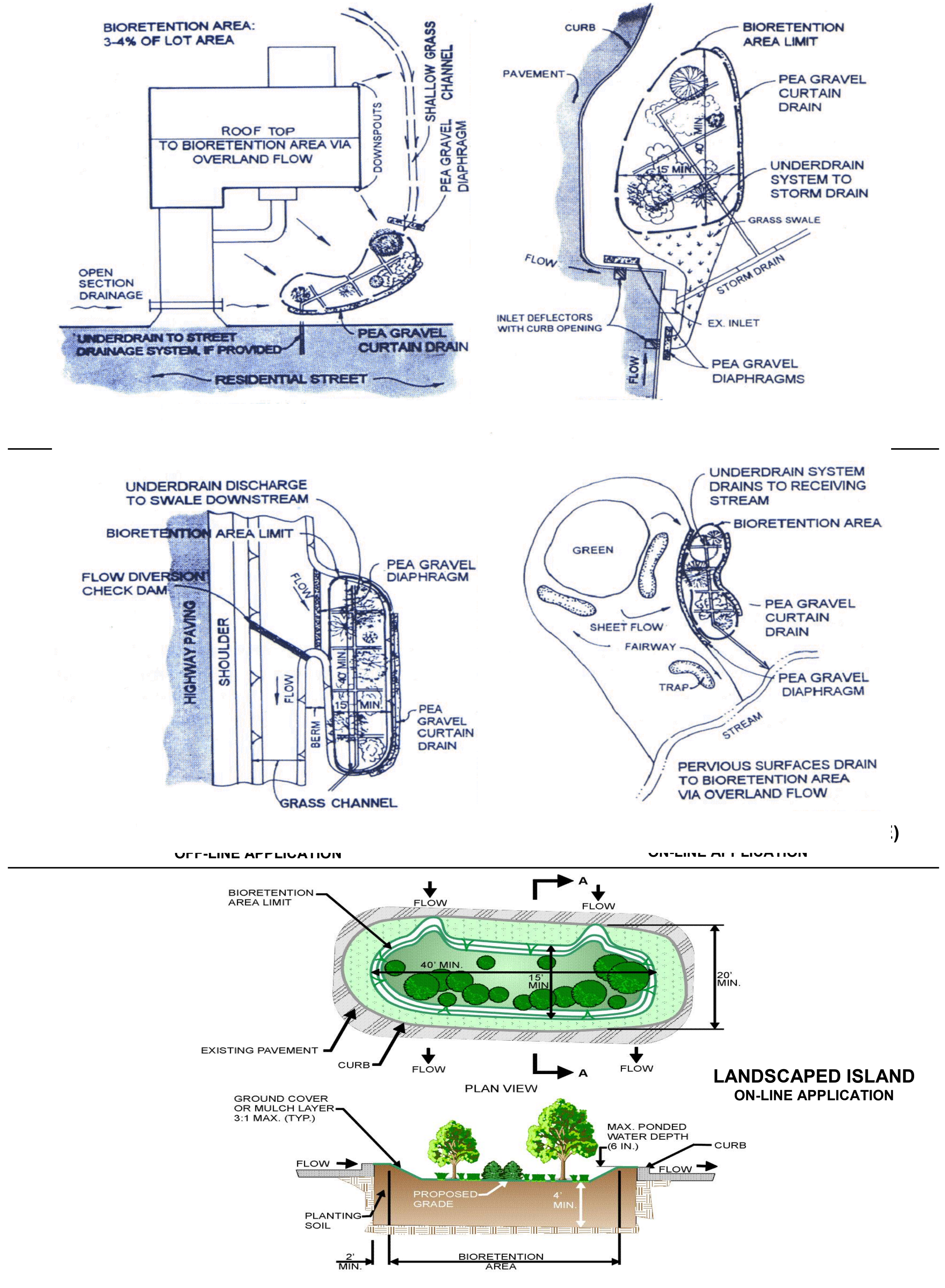


Figure 2.2 Bioretention Area Applications
(Source: Claytor and Schueler, 1996)

2.2 Stormwater Management Suitability

Bioretention areas are designed primarily for stormwater quality, i.e. the removal of stormwater 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 stormwater 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 2.3 of this section). 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 stormwater 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 2.3 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 1.0).

2.3 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 Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

2.4 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 stormwater 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 stormwater quality treatment capacity.

The following criteria should be evaluated to ensure the suitability of a bioretention area for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO
- Hot Spot - YES

Physical Feasibility - Physical Constraints at Project Site

- <u>Drainage Area</u> – 5 acres maximum; 0.5 to 2 acres are preferred.
- <u>Space Required</u> – Approximately 5-7% of the tributary impervious area is normally required.
- <u>Site Slope</u> – No more than 6% slope
- <u>Minimum Head</u> – Elevation difference needed at a site from the inflow to the outflow: 3-5 feet
- <u>Minimum Depth to Water Table</u> – A separation distance of 2 feet recommended between the bottom of the bioretention facility and the elevation of the seasonally high water table.
- <u>Soils</u> – No restrictions; engineered media required

Other Constraints / Considerations

- <u>Aquifer Protection</u> – Do not allow infiltration of filtered hotspot runoff into groundwater

2.5 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. Stormwater flows greater than the WQ_v are diverted to other controls or downstream (see *Section 1.4.2* 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:

- Grass filter strip (or grass channel) between the contributing drainage area and the ponding area, except where site conditions preclude its use,
- Ponding area containing vegetation with a planting soil bed,
- Organic/mulch layer,
- Pea gravel layer between the planting soil and the gravel underneath to provide filtering of the particles prior to entering gravel layer,
- 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:

- Optional **sand filter layer** to spread flow, filter runoff, and aid in aeration and drainage of the planting soil.
- **Stone diaphragm** at the beginning of the grass filter strip to reduce runoff velocities and spread flow into the grass filter.
- **Inflow diversion** or an **overflow structure** consisting of one of five main methods:
  1) Use a flow diversion structure
  2) For curbed pavements use an inlet deflector (see Figure 2.6).
  3) 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 2.5).
  4) Figure 2.2 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.
  5) An in-system overflow consisting of an overflow catch basin inlet and/or a pea gravel curtain drain overflow.

See Figure 2.3 for an overview of the various components of a bioretention area. Figure 2.4 provides a plan view and profile schematic of an on-line bioretention area. An example of an off-line facility is shown in Figure 2.5.

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 (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 the table below.

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)

- For on-line configurations, a grass filter strip with a pea gravel diaphragm is typically utilized (see Figure 2.3) 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 *Section 13.0* (*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 *Section 4*.0 (*Grass Channel)*.
- The mulch layer should consist of 2 to 4 inches of commercially available 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 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. The filter fabric should be non-woven with a minimum permittivity rate of 75 to 125 gal/min/ft^2. Additional geotextile properties are provided in the table below.

Filter Fabric Properties		
Geotextile Property	**Value**	**Test Method**
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

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 catch basin (Figure 2.3), 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 *Section 1.5.2 of the Landscape Technical Manual*.

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)

2.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Protection Volume (WQ_v), Streambank Protection Volume (SP_v), and the flood mitigation storm Flood Discharge (Q_f).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual*.

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 *Sections 2.4 and 2.5* (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 2.5.10* (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 *Section 1.4 of the Water Quality Technical Manual*).

(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):

$$\mathbf{A_f = (WQ_v)\,(d_f)\,/\,[\,(k)\,(h_f + d_f)\,(t_f)]} \qquad \mathbf{(2.1)}$$

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 *Section 2.5.3* (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 *Section 2.5.3* (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 *Section 3.6* (Landscaping) and *Section 1.5.2 of the Landscape Technical Manual* for more details.

See *Section 29.3* for a Bioretention Area Design Example

2.7 Inspection and Maintenance Requirements

Table 2.1 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.

2.8 Example Schematics

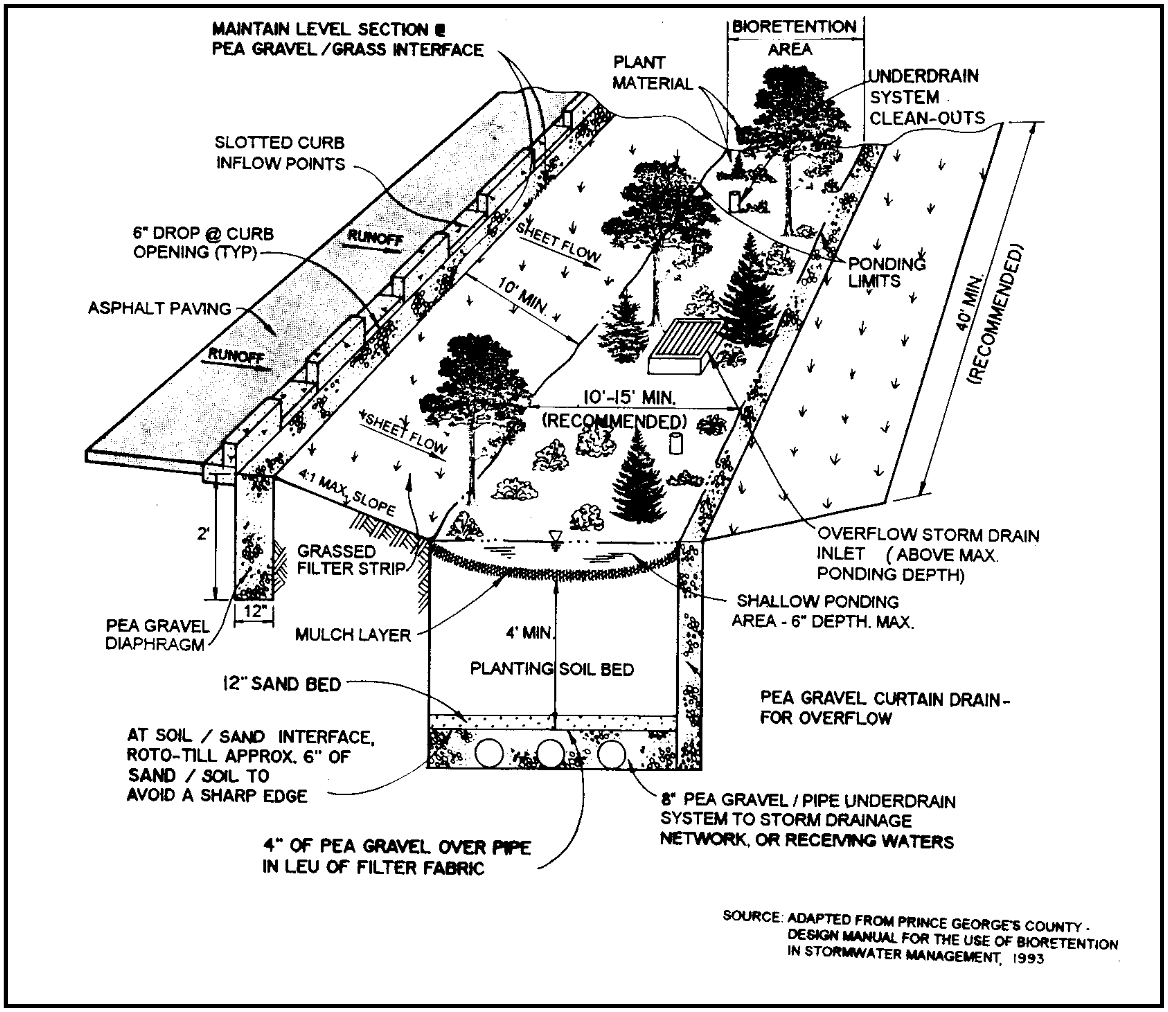


Figure 2.3 Schematic of a Typical Bioretention Area
(Source: Claytor and Schueler, 1996)

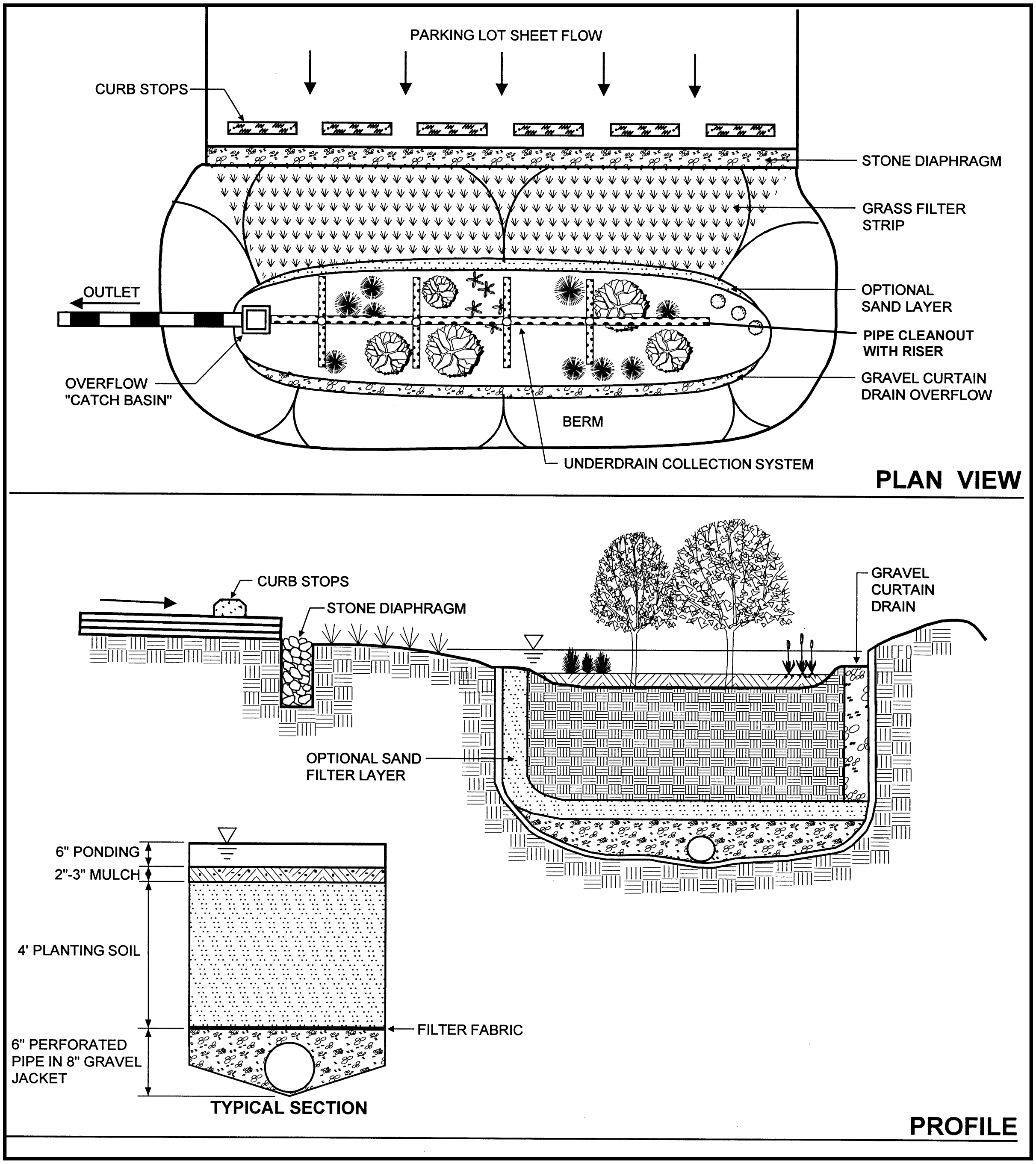


Figure 2.4 Schematic of a Typical On-Line Bioretention Area
(Source: Claytor and Schueler, 1996)

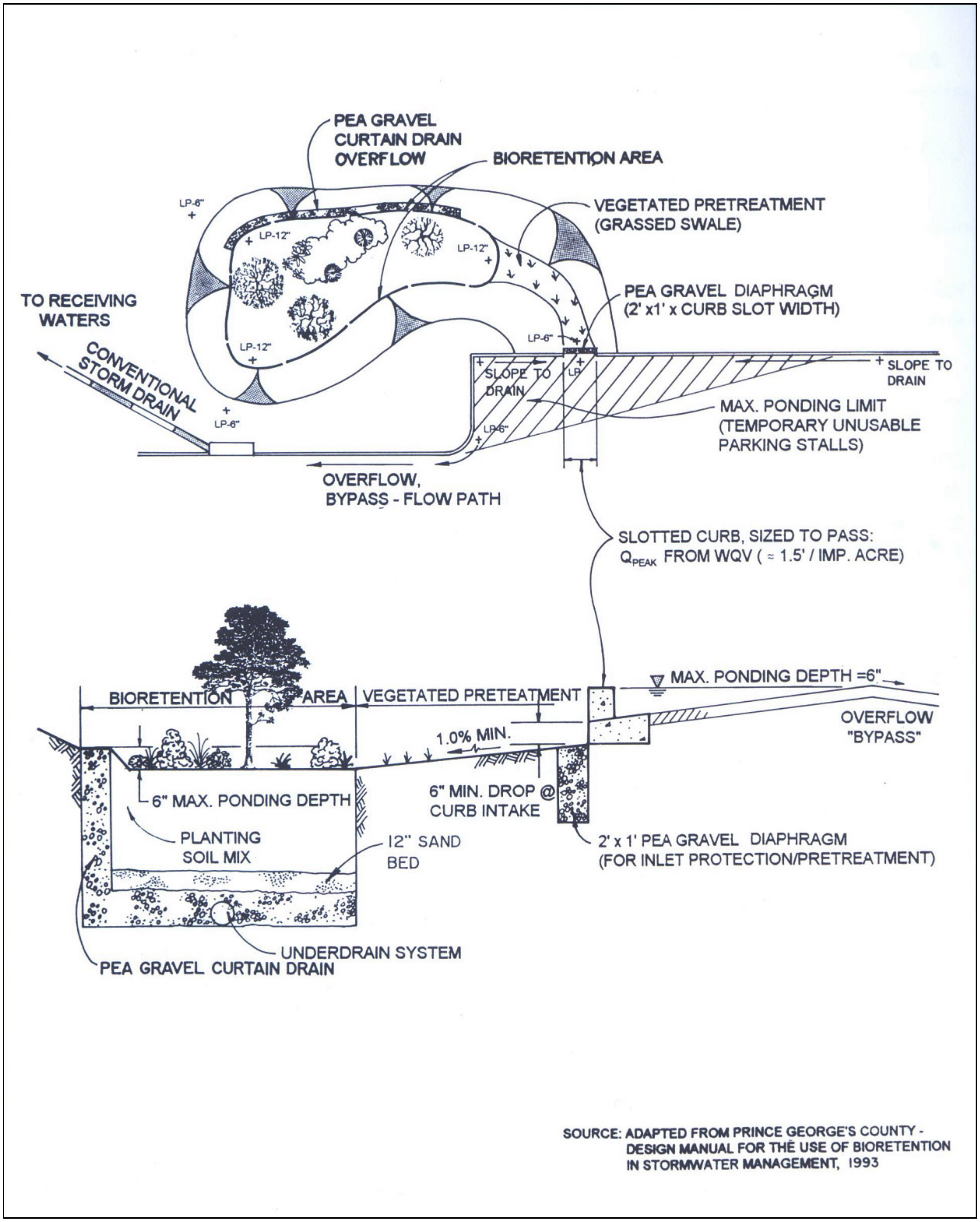


Figure 2.5 Schematic of a Typical Off-Line Bioretention Area
(Source: Claytor and Schueler, 1996)

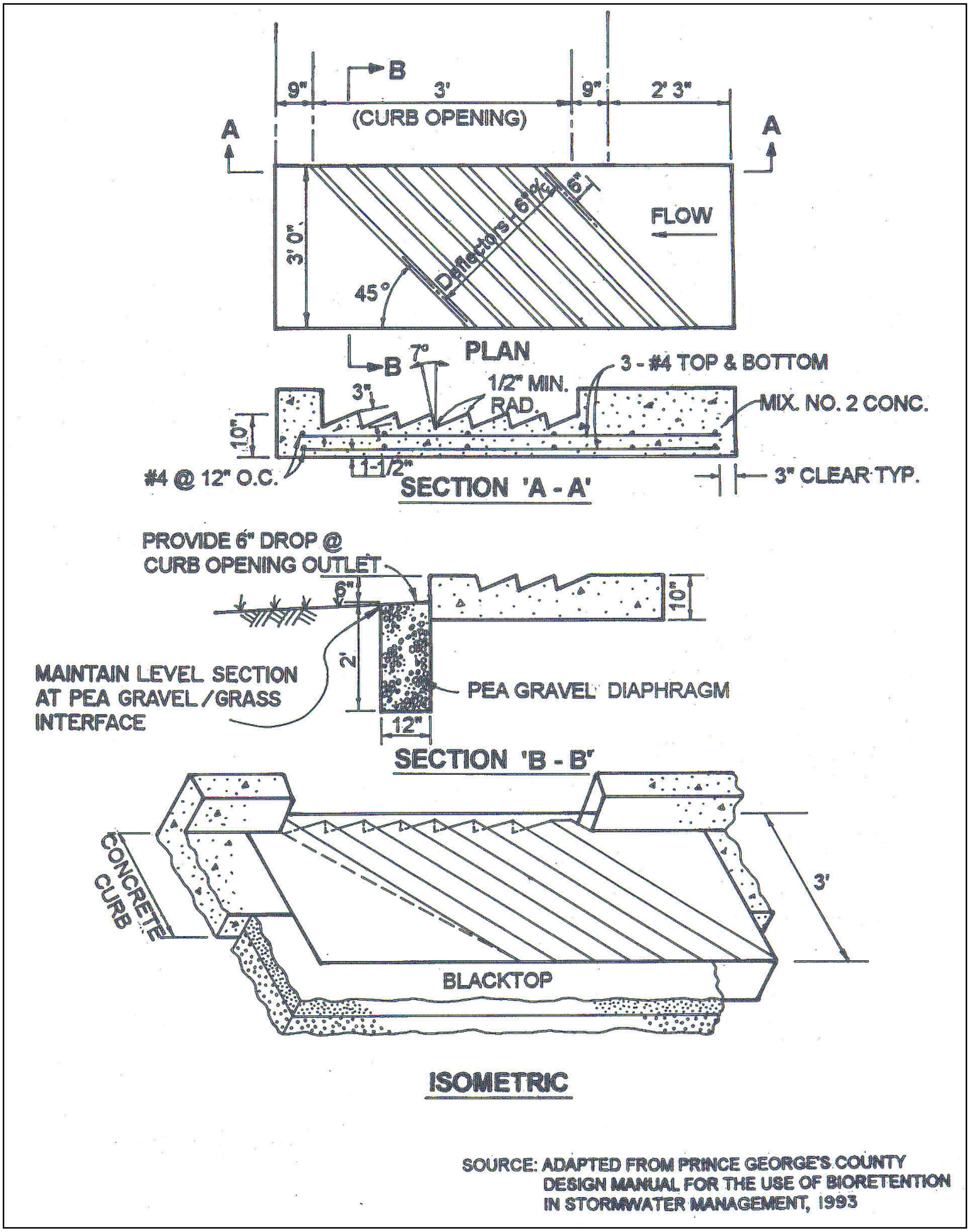


Figure 2.6 Schematic of a Typical Inlet Deflector
(Source: Claytor and Schueler, 1996)

Table 2.2 Design Procedure Form: Bioretention Areas

Design Procedure Form: Bioretention Areas

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = ________
Compute WQ_v	WQ_v = ________ acre-ft
1b. Compute SP_v	SP_v = ________ acre-ft
Compute average release rate	release rate = ________ cfs
Compute (as necessary) Q_f	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: ____

____

3.0 Enhanced Swales

General Application
Structural Stormwater Control

Description: Vegetated open channels that are explicitly designed and constructed to capture and treat stormwater 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 the Criteria Manual

ADVANTAGES / BENEFITS:

- Combines stormwater 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

STORMWATER 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

3.1 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 stormwater 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 3.1 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 stormwater 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 *Sections 4.0 and 13.0 of the Site Development Controls Technical Manual*, respectively.

3.2 Stormwater Management Suitability

Enhanced swale systems are designed primarily for stormwater 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 stormwater contaminants. Wet swales achieve pollutant removal both from sediment accumulation and biological removal.

Section 3.3 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.

3.3 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

3.4 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 stormwater pond for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – NO
- Regional Stormwater 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

3.5 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 3.2 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 3.3 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 flood mitigation storm) 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 *Section 1.0 of the Landscape Technical Manual.*

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 the *Landscape Technical Manual*
- 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 *Section 4.0 of the Hydrology Technical Manual* 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

3.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

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 Focus Areas are found in *Section 1.0 of the Planning Technical Manual.*

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 *Sections 3.4 and 3.5* (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 3.5* (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 *Section 3.5* (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 *Section 3.5* (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 *Section 3.5* (Landscaping) and the *Landscape Technical Manual* for more details.

See *Section 29.6* for an Enhanced Swale Design Example

3.7 Inspection and Maintenance Requirements

Table 3.1 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.

3.8 Example Schematics

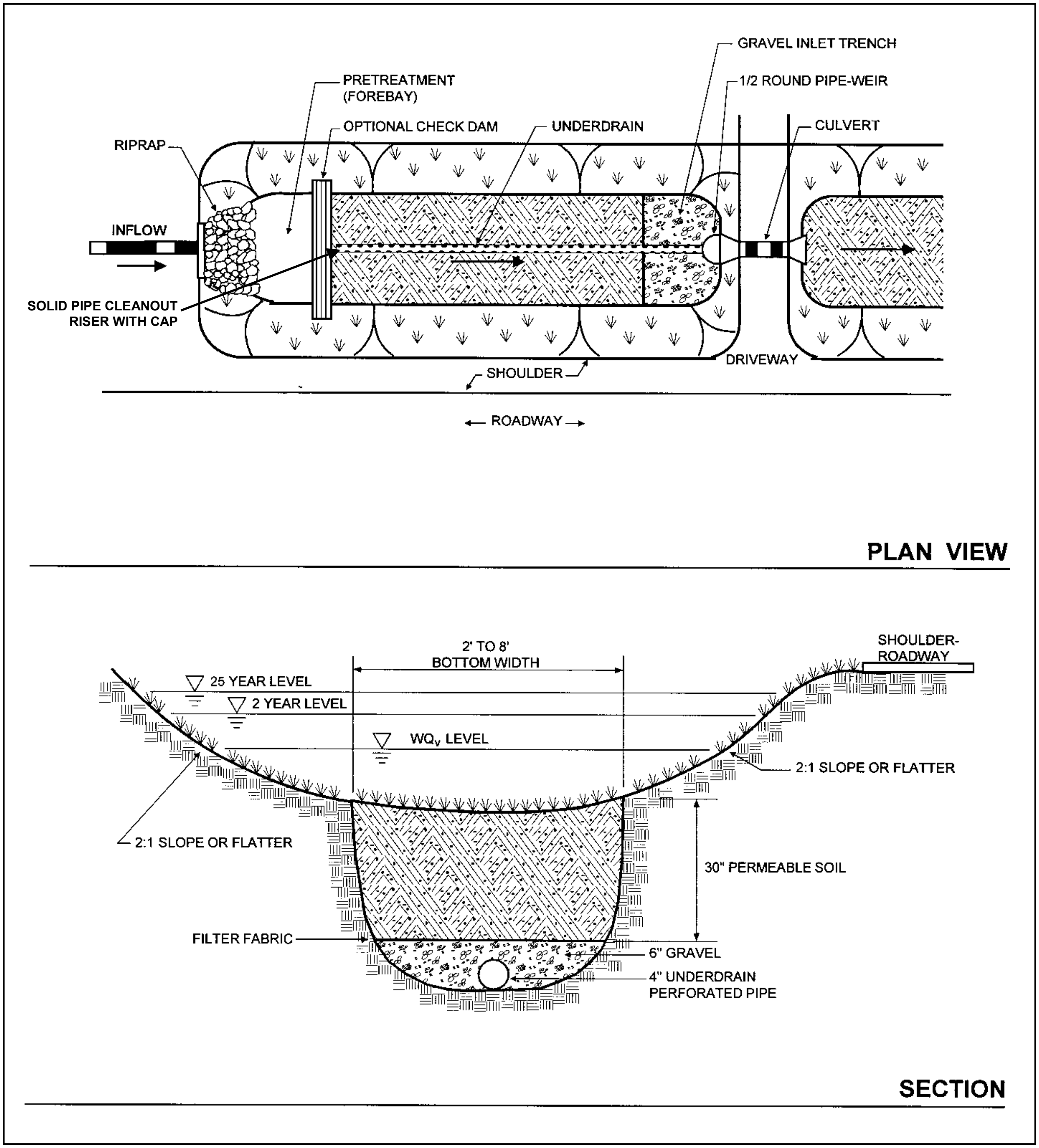


Figure 3.2 Schematic of Dry Swale
(Source: Center for Watershed Protection)

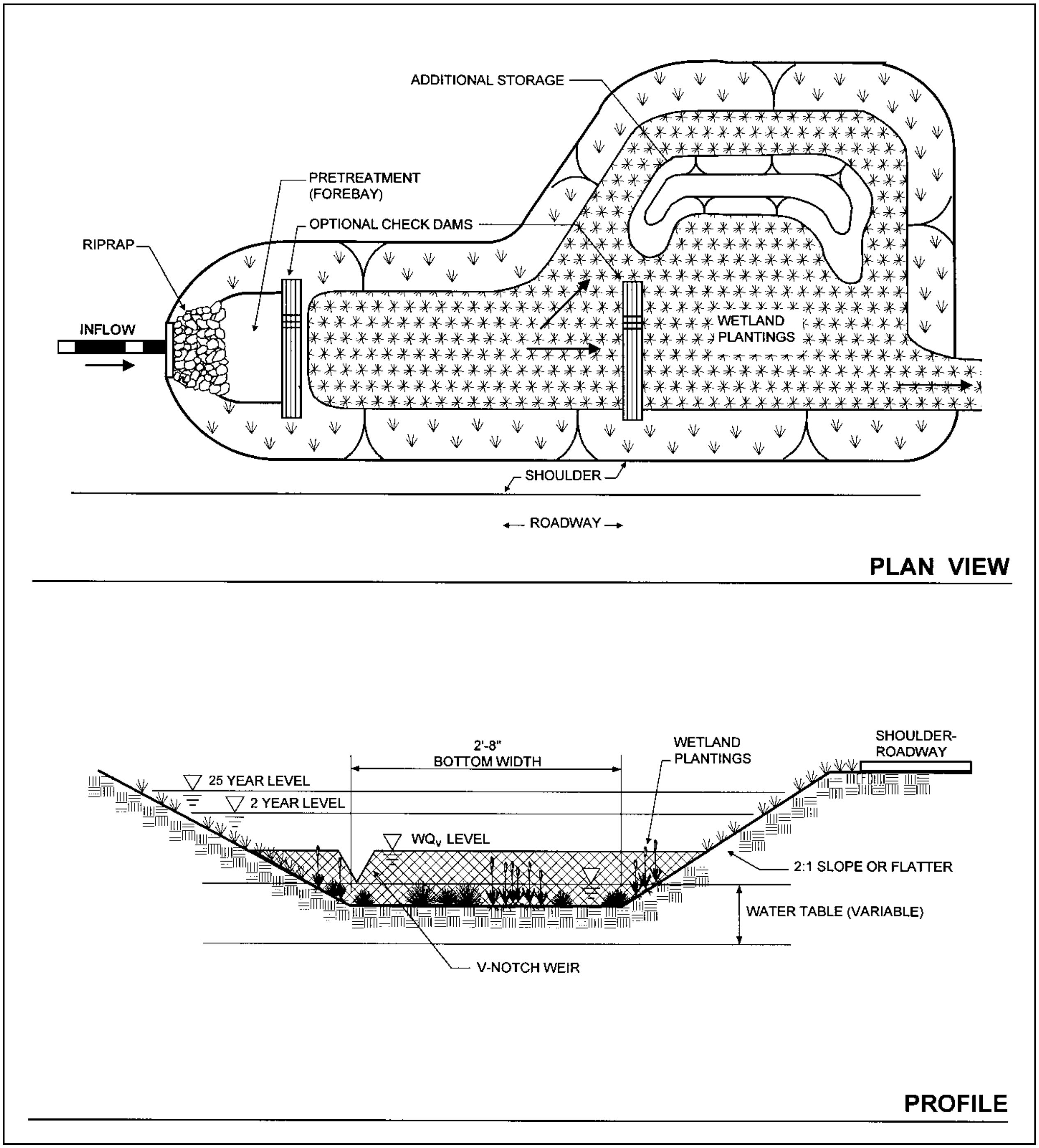


Figure 3.3 Schematic of Wet Swale
(Source: Center for Watershed Protection)

3.9 Design Forms

Design Procedure Form: Enhanced Swales

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = ____________
Compute WQ_v	WQ_v = ____________ acre-ft
1b. Compute SP_v	SP_v = ____________ acre-ft
Compute average release rate	release rate = ____________ cfs
Compute Q_p (100-year detention volume required)	Q_p = ____________ acre-ft
Compute (as necessary) Q_f	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	V_{min} = ____________ fps
Requires separate computer analysis for velocity	
Overflow wier (use weir equation)	Weir Length = ____________ ft
Use weir equation for slot length ($Q = CLH^{3/2}$)	
9. Design low flow orifice at headwall	
Area of orifice from orifice equation	Area = ____________ ft^2
$Q = CA(2gh)^{0.5}$	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: ____

4.0 Grass Channel

Structural Stormwater Control

Description: Vegetated open channels designed to filter stormwater 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

STORMWATER 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

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

4.1 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.

4.2 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.

4.3 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 1.0 of the Water Quality Technical Manual*).

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 the *Landscape Technical Manual* for a list of appropriate grasses for use in North Central Texas.

See *Section 3.2 of the Hydraulics Technical Manual* 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 4.1 provides sizing guidance for grass channels for a 1-acre drainage area. The minimum grassed channel length should be 20 feet.

Table 4.1 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)

4.4 Inspection and Maintenance Requirements

Table 4.2 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)

4.5 Example Schematics

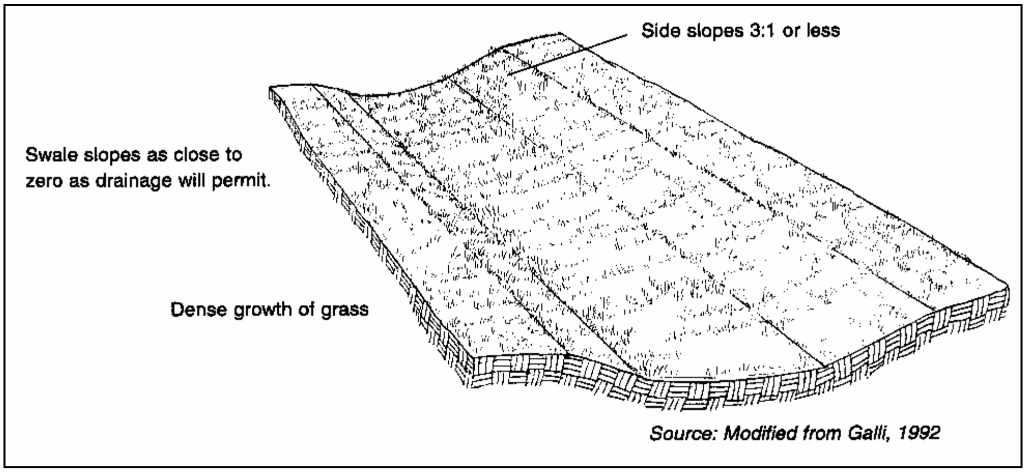


Figure 4.1 Typical Grass Channel

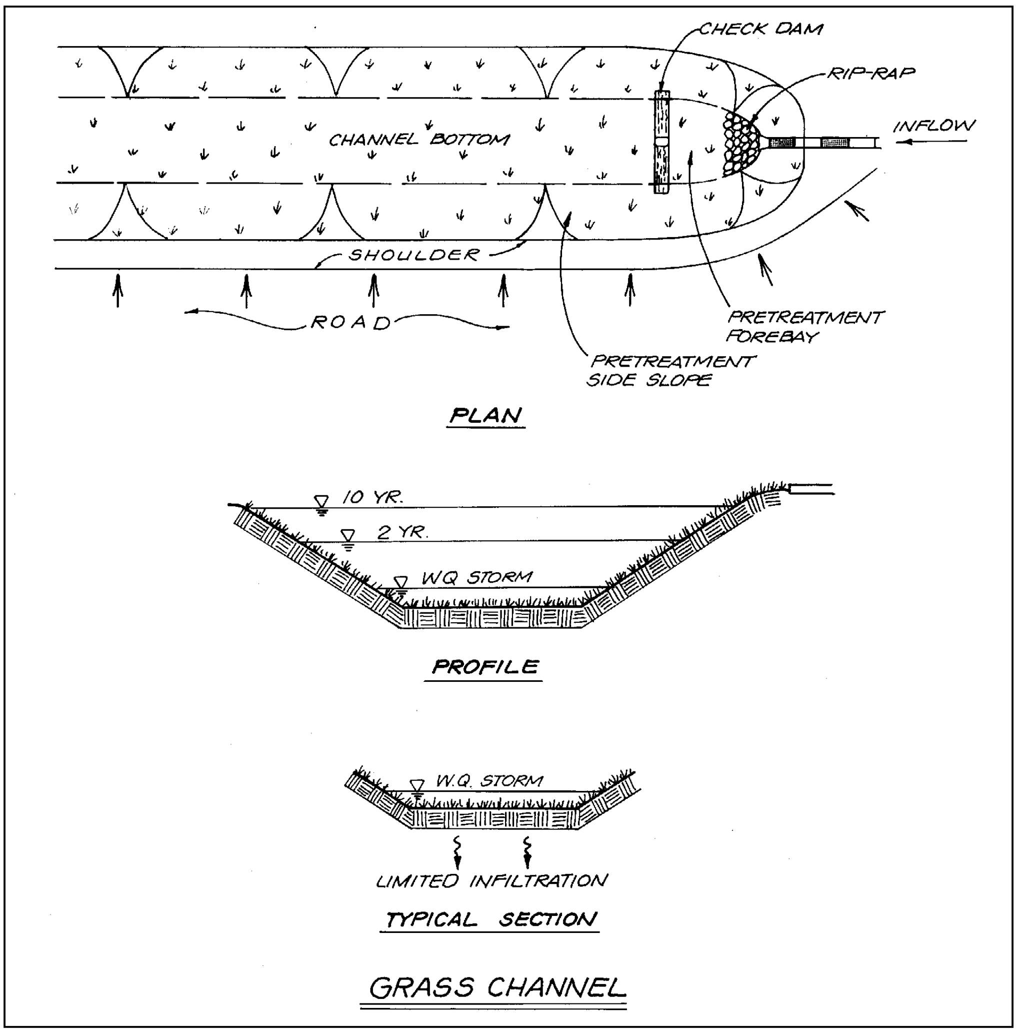


Figure 4.2 Schematic of Grass Channel

4.6 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 Section 1.0 of the Water Quality Technical Manual 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):

$$\begin{aligned} CN &= 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}] \\ &= 1000/[10+5*1.5+10*0.82-10(0.82^2+1.25*0.82*1.5)^{½}] \\ &= 92.4 \qquad \text{(Use CN = 92)} \end{aligned}$$

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 1.11 of the Hydrology Technical Manual*, I_a = 0.174, therefore I_a/P = 0.174/1.5 = 0.116.

From *Figure 1.10 of the Hydrology Technical Manual* 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

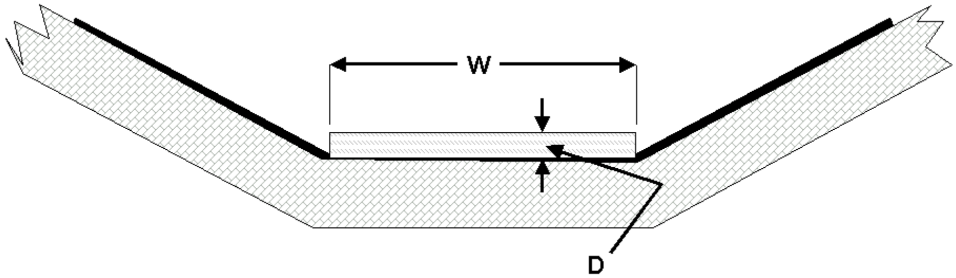
Then:

$Q_{wq} = Q = VA = 1.49/n\ D^{2/3}\ S^{1/2}\ DW$

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 3.2 of the Hydraulics Technical Manual* to complete the grass channel design for a specified design storm event.

5.0 Open Conveyance Channel

Stormwater 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

STORMWATER 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

5.1 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 stormwater 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 4.0* and *Section 3.0*, 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 5.1 Open Channel Examples

5.2 Pollutant Removal Capabilities

Open conveyance channels or drainage ditches are designed for conveyance purposes only. For open channels with pollutant capabilities, refer to *Section 4.0* and *Section 3.0*, respectively.

5.3 Design Criteria and Specifications

Detailed design criteria and specifications, as prepared by the Federal Highway Administration, are presented in *Section 3.2 of the Hydraulics Technical Manual*. 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 flood mitigation storm event.

5.4 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.

6.0 Alum Treatment System

Structural Stormwater Control

Description: Chemical treatment of stormwater 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 stormwater treatment from a piped stormwater drainage system
- High maintenance requirements
- Alum application will lower pH of receiving waters
- High capital and operations and maintenance costs

STORMWATER 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

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

6.1 General Description

The process of alum (aluminum sulfate) treatment provides treatment of stormwater runoff from a piped stormwater 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 stormwater 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 stormwater treatment facilities in Florida have ranged from $135,000 to $400,000. The capital construction costs of alum stormwater 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 stormwater 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.

6.2 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 stormwater 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%

6.3 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.

6.4 Inspection and Maintenance Requirements

Table 6.1 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 6.1 Alum Treatment System and Injection Equipment

7.0 Culverts

Stormwater Control

Description: A short, closed (covered) conduit that conveys stormwater 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

STORMWATER 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

7.1 General Description

A culvert is a short, closed (covered) conduit that conveys stormwater 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.

7.2 Pollutant Removal Capabilities

Culverts are designed for stormwater conveyance purposes and do not provide pollutant removal capabilities.

7.3 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 7.1) 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 7.2. See *Section 3.3 of the Hydraulics Technical Manual* for detailed culvert design procedures and instruction.

7.4 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.

7.5 Example Schematic

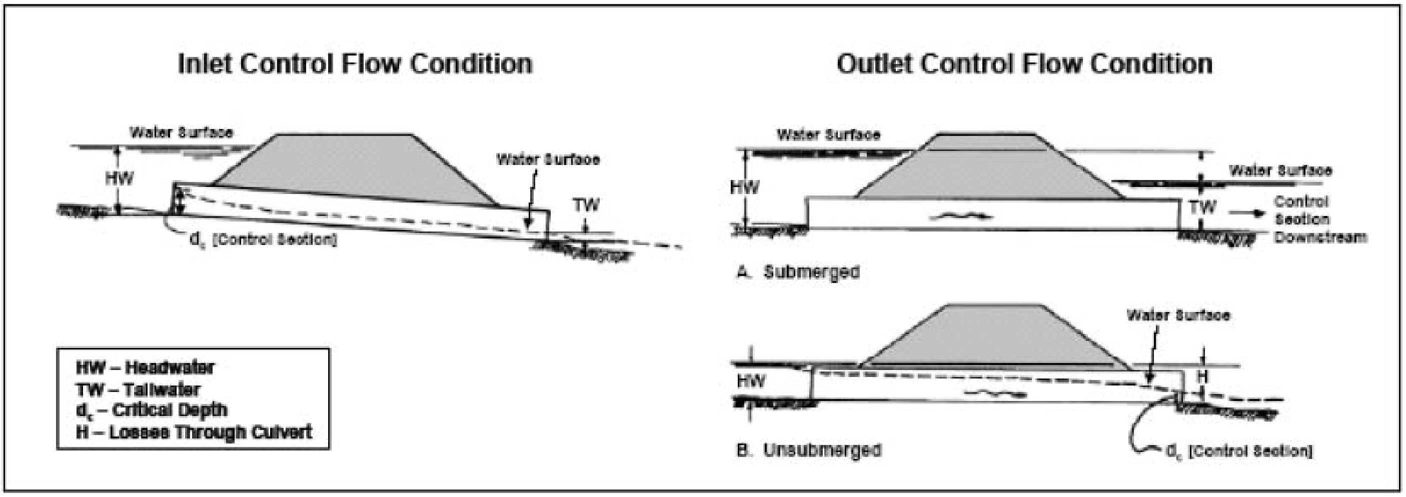


Figure 7.1 Culvert Flow Conditions

Figure 7.2 Culvert Examples

8.0 Inlets

Stormwater 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 stormwater 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.

STORMWATER 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

8.1 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).

8.2 Pollutant Removal Capabilities

Although inlets prevent large debris from passing to the storm sewer system, they are designed for stormwater conveyance purposes and do not provide pollutant removal capabilities.

8.3 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 3.3 of the Hydraulics Technical Manual* for detailed inlet design procedures and instruction.

8.4 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 stormwater 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 municipalities 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 8.1 Example Inlets

9.0 Pipe Systems

Stormwater Control

Description: Pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at structural stormwater 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

STORMWATER 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

9.1 General Description

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used in the minor stormwater drainage system for transporting runoff from roadway and other inlets to outfalls at structural stormwater 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.

9.2 Pollutant Removal Capabilities

The stormwater pipe system is designed for conveyance purposes and does not provide pollutant removal capabilities.

9.3 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 1.27 of the Hydraulics Technical Manual*) 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 1.2 of the Hydraulics Technical Manual* for detailed pipe system design procedures and instruction.

9.4 Inspection and Maintenance Requirements

Maintaining stormwater 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

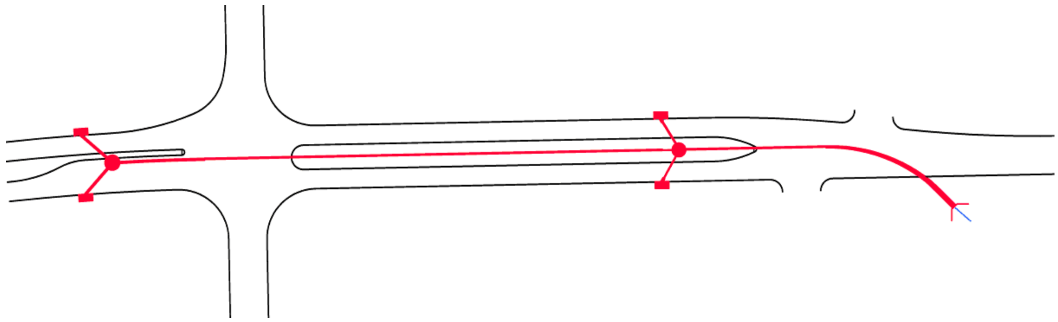
- Regularly inspect and clean up hot spots and other storm drainage areas where illegal dumping and disposal occurs.
- Establish a system for tracking incidents. The system should be designed to identify the following:
    - Illegal dumping hot spots
    - Types and quantities (in some cases) of wastes
    - 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.

9.5 Example Schematic



10.0 Dry Detention / Extended Detention Dry Basins

Detention
Structural Stormwater Control

Description: A surface storage basin or facility designed to provide water quantity control through detention and/or extended detention of stormwater 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 stormwater (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 stormwater 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

STORMWATER 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

10.1 General Description

Dry detention and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. These facilities temporarily detain stormwater 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 (flood mitigation 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 1.0*).

Compatible multi-objective use of dry detention facilities in strongly encouraged.

10.2 Design Criteria and Specifications

Location

Dry detention and extended detention dry basins are to be located downstream of other structural stormwater 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 1.0* 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 flood mitigation 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 flood mitigation storm event.

  Routing calculations must be used to demonstrate that the storage volume and outlet structure configuration are adequate. See *Section 2.0 of the Hydraulics Technical Manual* for procedures on the design of detention storage.

- Storage may be subject to the requirements of the Texas Dam Safety Program (see *iSWM Program Guidance – Dams and Reservoirs in Texas*) 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 *iSWM Program Guidance – Dams and Reservoirs in Texas*).
- 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 2.2 of the Hydraulics Technical Manual* 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 4.0 of the Hydraulics Technical Manual*, for more guidance.
- An emergency spillway is to be included in the stormwater 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 *iSWM Program Guidance - Dams and Reservoirs in Texas*) 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.

10.3 Inspection and Maintenance Requirements

Table 10.1 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

10.4 Example Schematics

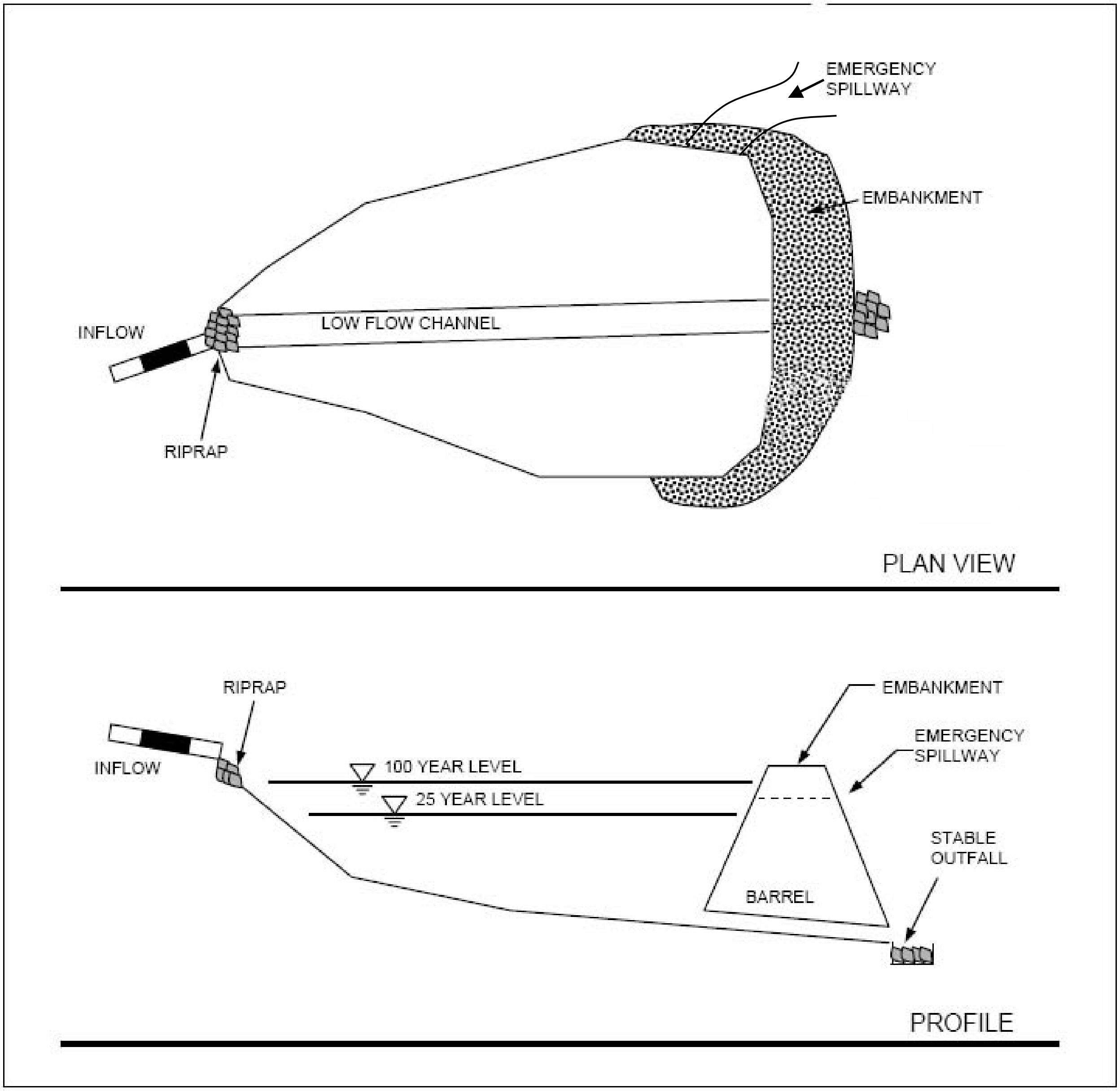


Figure 10.1 Schematic of Dry Detention Basin

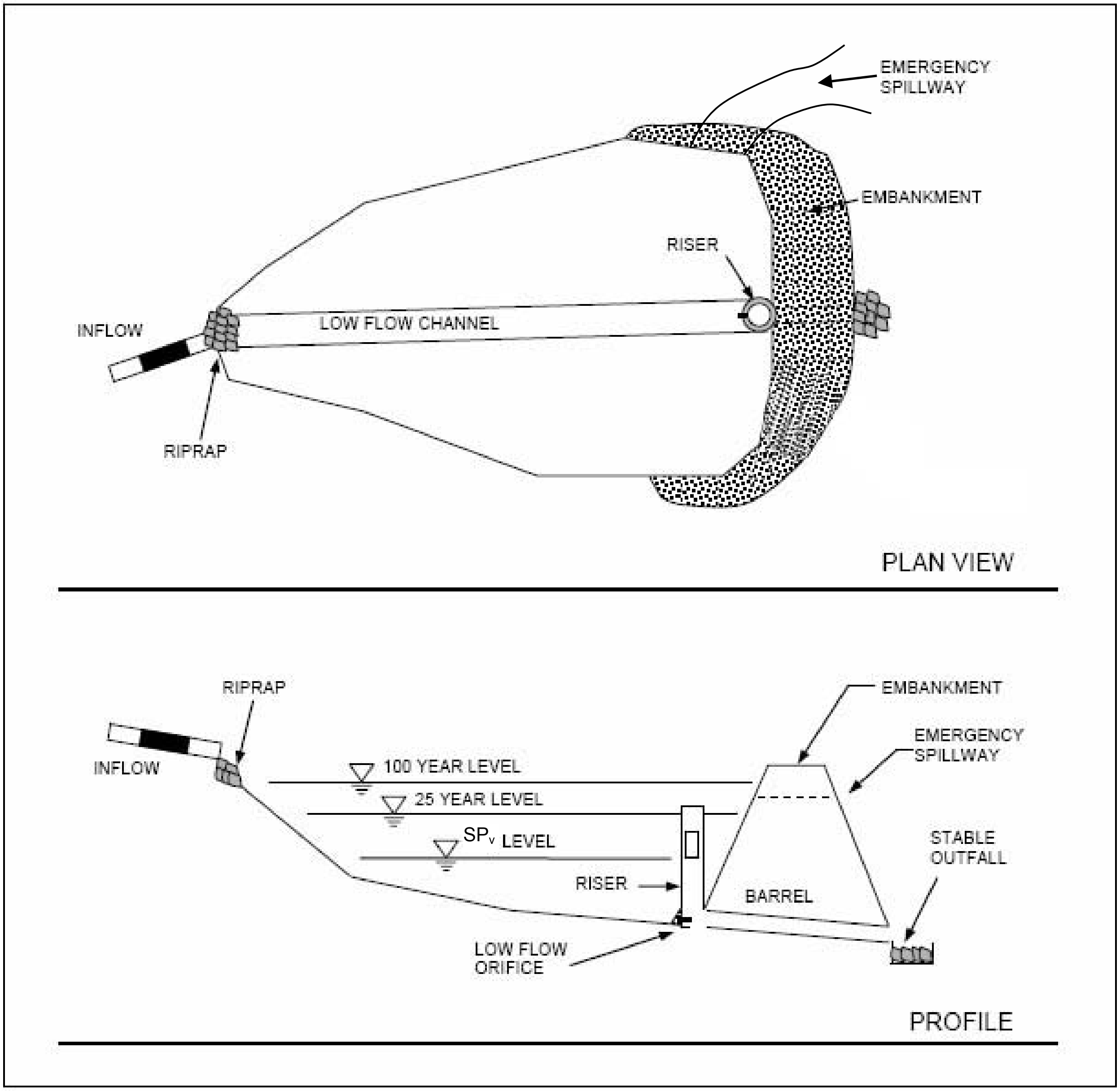


Figure 10.2 Schematic of Dry Extended Detention Basin

11.0 Multi-Purpose Detention Areas

Structural Stormwater Control

Description: A facility designed primarily for another purpose, such as parking lots and rooftops that can provide water quantity control through detention of stormwater 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 stormwater quantity only – not intended to provide water quality protection
- Localized flooding of area as intended may lead to property damage and additional liability

STORMWATER 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

11.1 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 stormwater 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 1.6*).

11.2 Design Criteria and Specifications

Location

Multi-purpose detention areas can be located upstream or downstream of other structural stormwater controls providing treatment of the water quality protection volume (WQ_v). See the *Section 1.6* 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 flood mitigation storm, if required.

Routing calculations must be used to demonstrate that the storage volume is adequate. See *Section 2.0 of the Hydraulics Technical Manual* 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 flood mitigation storm. The maximum depth of ponding at a flow control structure is 12 inches for a flood mitigation 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 (flood mitigation storm) 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 stormwater 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 stormwater 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.

11.3 Inspection and Maintenance Requirements

Table 11.1 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)

12.0 Underground Detention

Detention
Structural Stormwater 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 stormwater 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 stormwater 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

STORMWATER 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

12.1 General Description

Detention vaults are box-shaped underground stormwater 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 stormwater 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 1.6*). 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.

12.2 Design Criteria and Specifications

Location

Underground detention systems are to be located downstream of other structural stormwater controls providing treatment of the water quality volume (WQ_v). See *Section 1.6* 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 2.0 of the Hydraulics Technical Manual* 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 2.2 of the Hydraulics Technical Manual* 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 4.0 of the Hydraulics Technical Manual*, for more guidance.
- A high flow bypass is to be included in the underground detention system design to safely pass the extreme flood flow.

12.3 Inspection and Maintenance Requirements

Table 12.1 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

12.4 Example Schematics

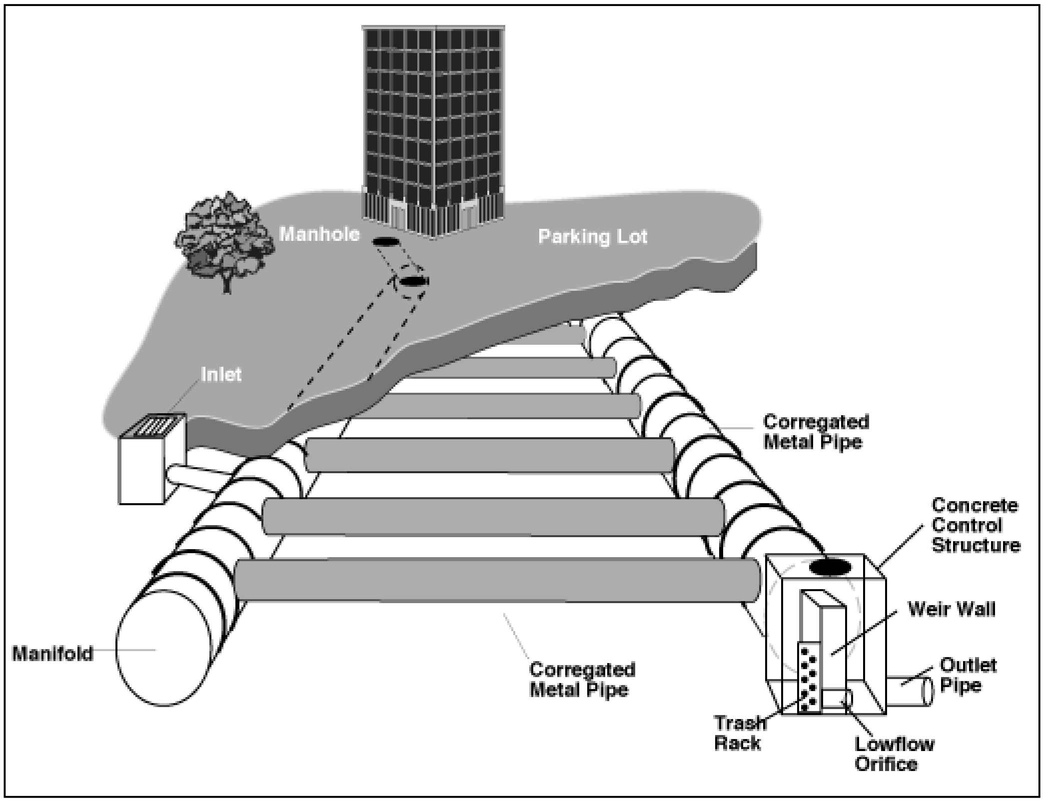


Figure 12.1 Example Underground Detention Tank System

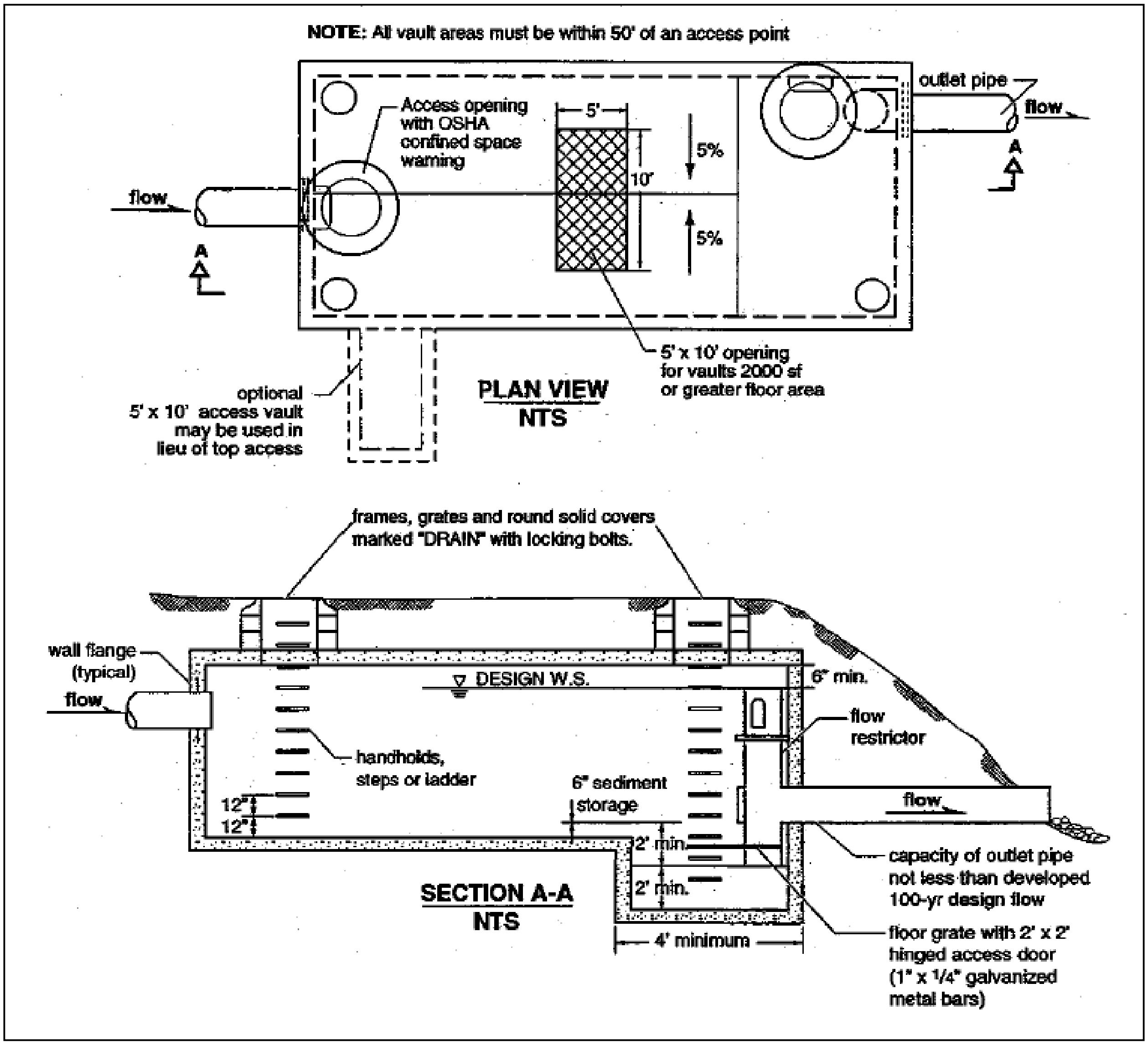


Figure 12.2 Schematic of Typical Underground Detention Vault
(Source: WDE, 2000)

13.0 Filter Strip

Structural Stormwater 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

STORMWATER 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

13.1 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 stormwater 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 13.0* for more information).

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. 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 stormwater runoff. Filter strips are typically an on-line practice, so they must be designed to withstand the full range of storm events without eroding.

13.2 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%

13.3 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 *Section 1.0 of the Landscape Technical Manual* 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:

$$q = \frac{0.023}{n} Y^{\frac{5}{3}} S^{\frac{1}{2}} \quad \textbf{(13.1)}$$

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:

$$W_{fMIN} = \frac{Q}{q} \quad \textbf{(13.2)}$$

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):

$$L_f = \frac{(T_t)^{1.25} (P_{2\text{-}24})^{0.625} (S)^{1/2}}{3.34n} \quad \textbf{(13.3)}$$

L_f = length of filter strip parallel to flow path (ft)

T_t = travel time through filter strip (minutes)

$P_{2\text{-}24}$ = 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 13.1 provides sizing guidance for bioretention filter strips for pretreatment.

Table 13.1 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)

13.4 Inspection and Maintenance Requirements

Table 13.2 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.

13.5 Example Schematic

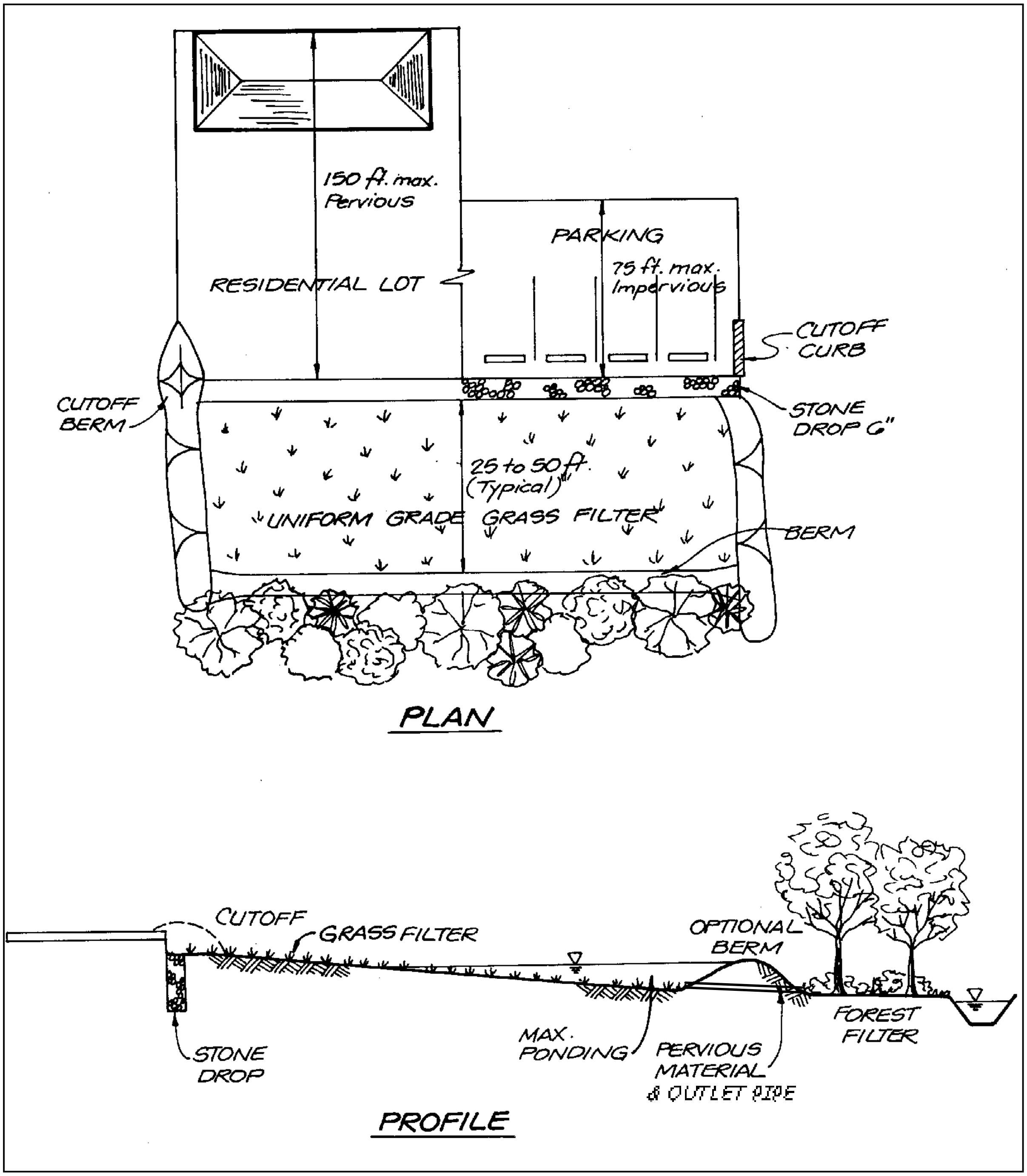


Figure 13.1 Schematic of Filter Strip (with Berm)

13.6 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 13.1:

$q = 0.0237/0.25 * (1.0)^{5/3} * (4)^{1/2} = 0.19$ cfs/ft

Water Quality Peak Flow

See *Section 1.4 of the Water Quality Technical Manual* for details

Compute the Water Quality Volume in inches:

WQv = 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(Q2+1.25*Q*P)½]

= 1000/[10+5*1.5+10*0.82-10(0.822+1.25*0.82*1.5)½]

= 92.4 (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 1.11 in the Hydrology Technical Manual*, I_a = 0.174, therefore I_a/P = 0.174/1.5 = 0.116.

From *Figure 1.10 in the Hydrology Technical Manual* for a Type II storm (using the limiting values) q_u = 950 csm/in, and therefore:

Q_{wq} = (950 csm/in) (0.34ac/640ac/mi^2) (1.02") = 0.51 cfs

Minimum Filter Width

Using Equation 13.2:

$W_{fMIN} = Q/q = 0.51/0.19 = 2.7$ 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 *Section 5.0 of the Hydrology Technical Manual*) = 0.17 in/hr or 0.17*24= 4.08 inches
- Use 5 minute travel (contact) time

Using Equation 13.3:

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 13.2.

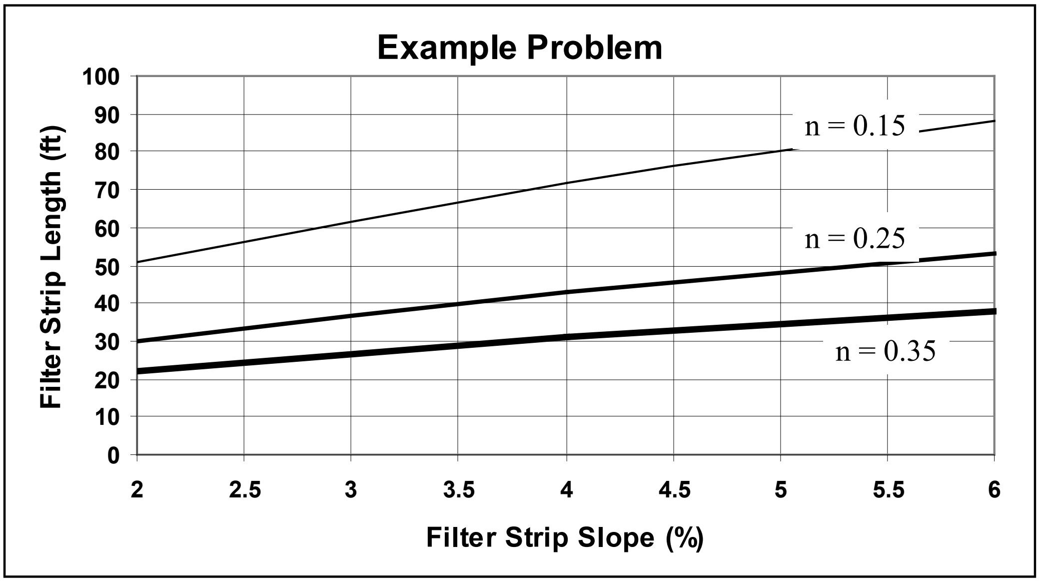


Figure 13.2 Example Problem Sensitivity of Filter Strip Length to Slope and Manning's n Values

Filter With Berm

- Pervious berm height is 6 inches

Compute the Water Quality Volume in cubic feet:

WQv = Rv * 1.5/12 * A = (0.05 + 0.009 * 70) * 1.5/12 * 0.34 = 0.029 Ac-ft or 1,259 ft3

For a berm height of 6 inches the "wedge" of volume captured by the filter strip is:

Volume = Wf * 0.5 * Lf * 0.5 = 0.25WfLf = 1,259 ft3

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.

14.0 Organic Filter

Structural Stormwater 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 stormwater 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

STORMWATER 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

14.1 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.

14.2 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%

14.3 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 14.1). 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. Stormwater 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 *Section 16.0* on *Sand Filters*) for the organic filter sizing and design steps.

14.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see *Section 16.0* on Sand Filters)

14.5 Example Schematic

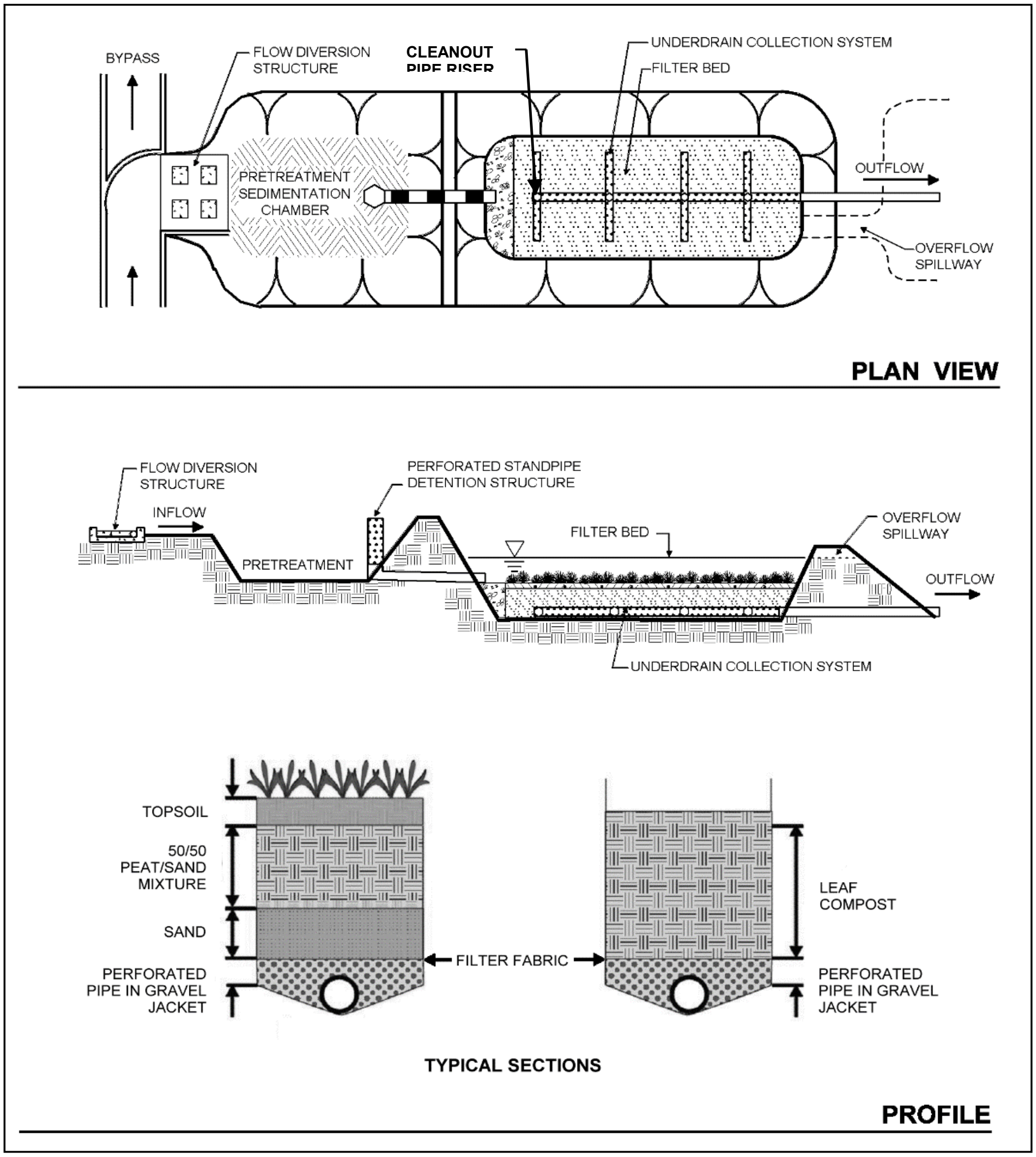


Figure 14.1 Schematic of Organic Filter
(Source: Center for Watershed Protection)

15.0 Planter Boxes

Structural Stormwater 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 stormwater 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 stormwater controls

STORMWATER 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

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

15.1 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 stormwater conveyance system.

15.2 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%).

15.3 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 stormwater.

Filter media should consist of sand, gravel and topsoil as shown in Figures 15.1-15.3. 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.

15.4 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 15.1 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

15.5 Example Schematics

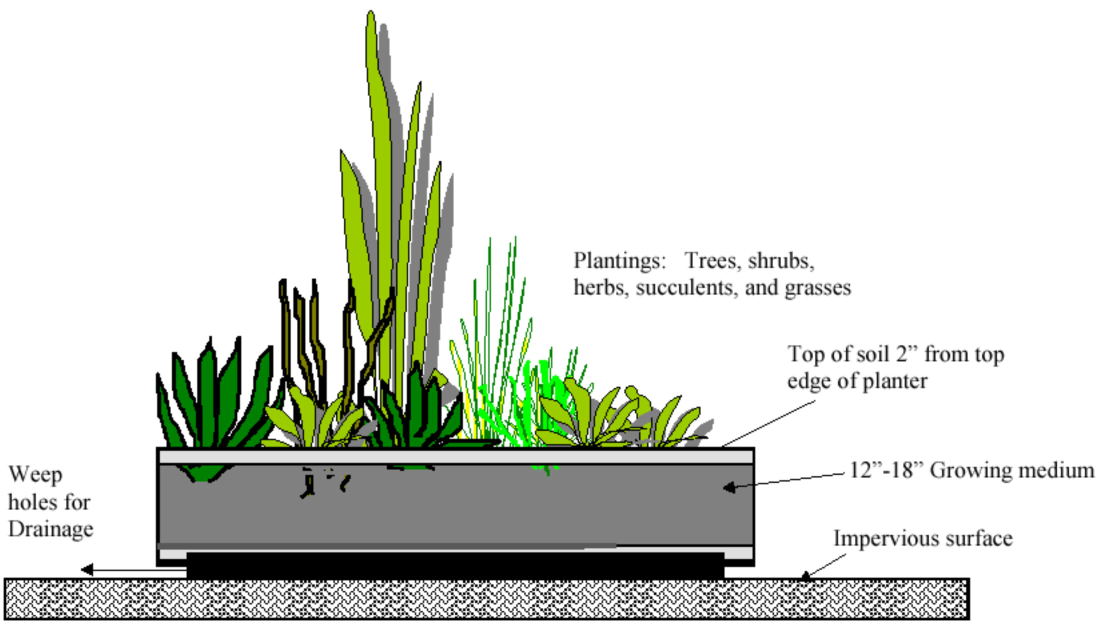


Figure 15.1 Schematic of Contained Planter Box
(Source: City of Portland, Oregon)

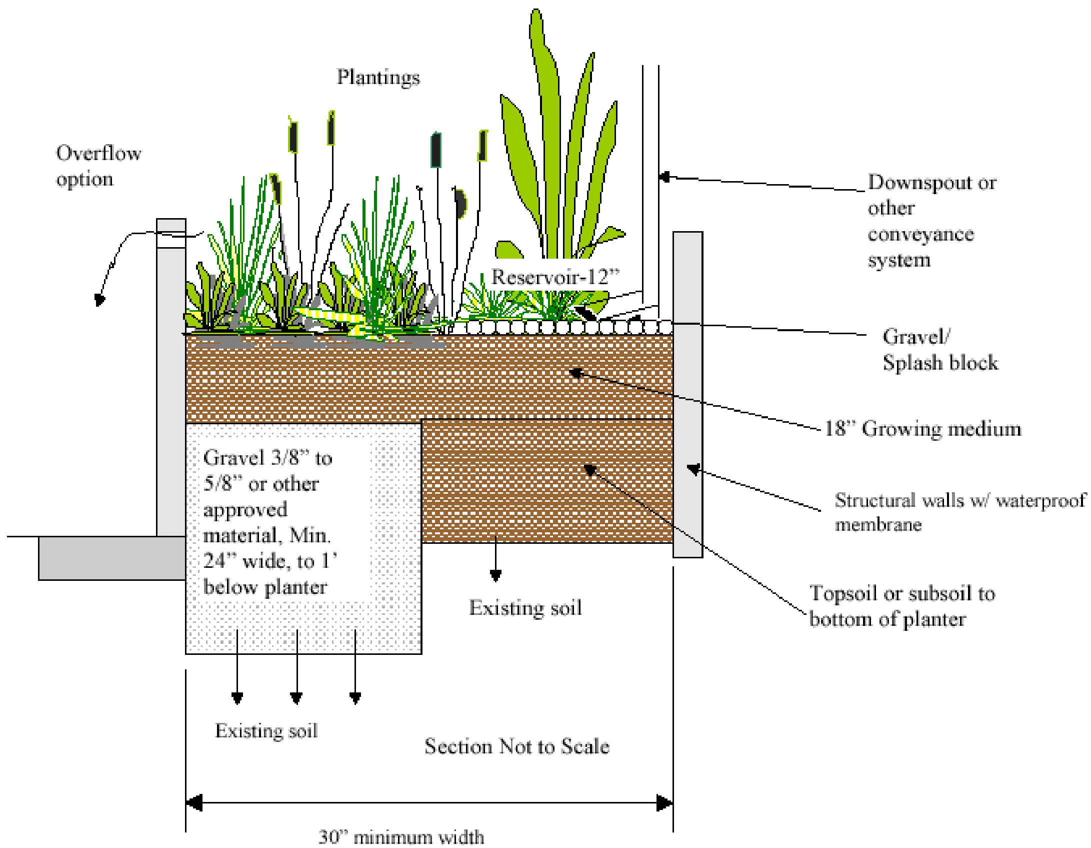


Figure 15.2 Schematic of Infiltration Planter Box
(Source: City of Portland, Oregon)

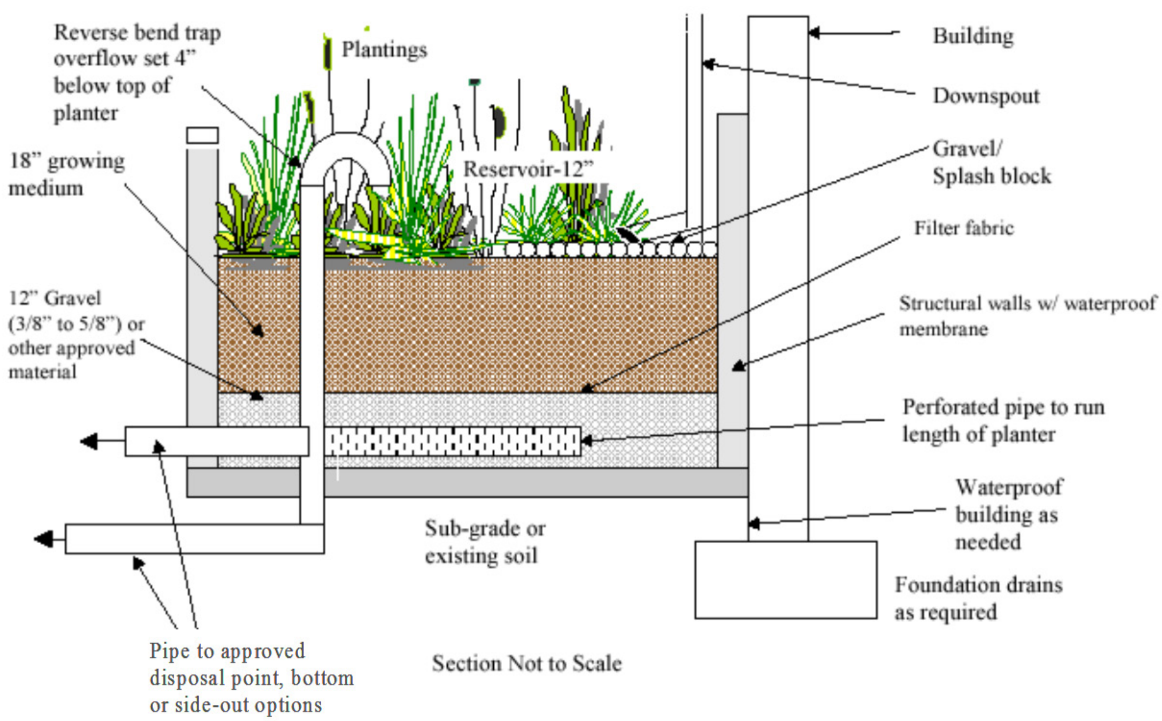


Figure 15.3 Schematic of Flow-Through Planter Box
(Source: City of Portland, Oregon)

16.0 Sand Filters

General Application
Structural Stormwater Control

Description: Multi-chamber structure designed to treat stormwater 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 stormwater 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

STORMWATER 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

16.1 General Description

Sand filters (also referred to as *filtration basins*) are structural stormwater controls that capture and temporarily store stormwater 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 *Section 16.0* on Underground Sand Filters for more details.

Surface Sand Filter

Perimeter Sand Filter

Figure 16.1 Sand Filter Examples

16.2 Stormwater Management Suitability

Sand filter systems are designed primarily as off-line systems for stormwater quality (i.e., the removal of stormwater 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, stormwater 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 16.3* 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 1.0).

16.3 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 Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

16.4 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 stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – NO
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater 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

16.5 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. Stormwater 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 16.6 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 16.7 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 16.2 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.

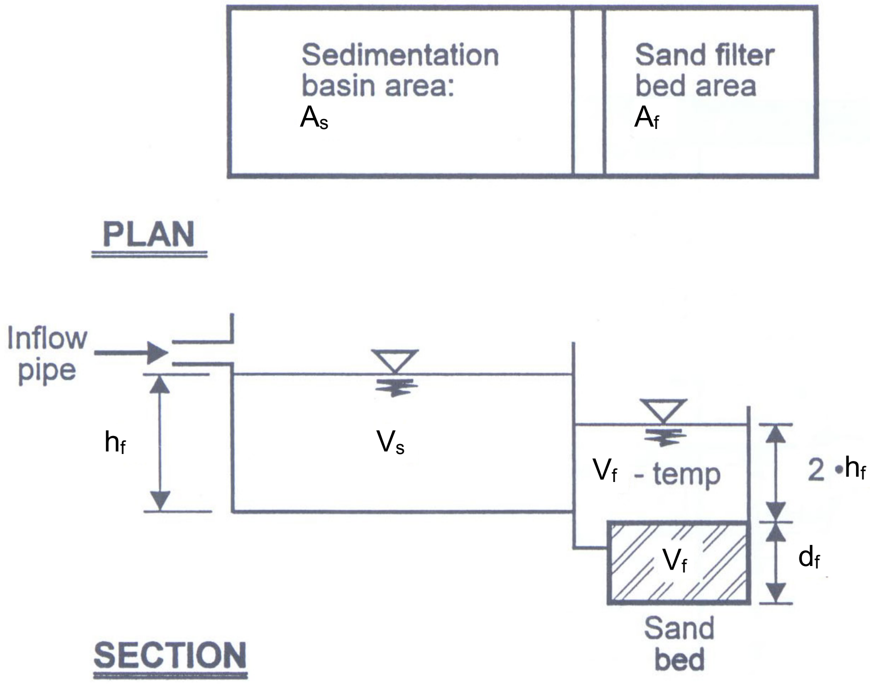


Figure 16.2 Surface Sand Filter Volumes
Source: Claytor and Schueler, 1996

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 16.4 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 16.3 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 16.4 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

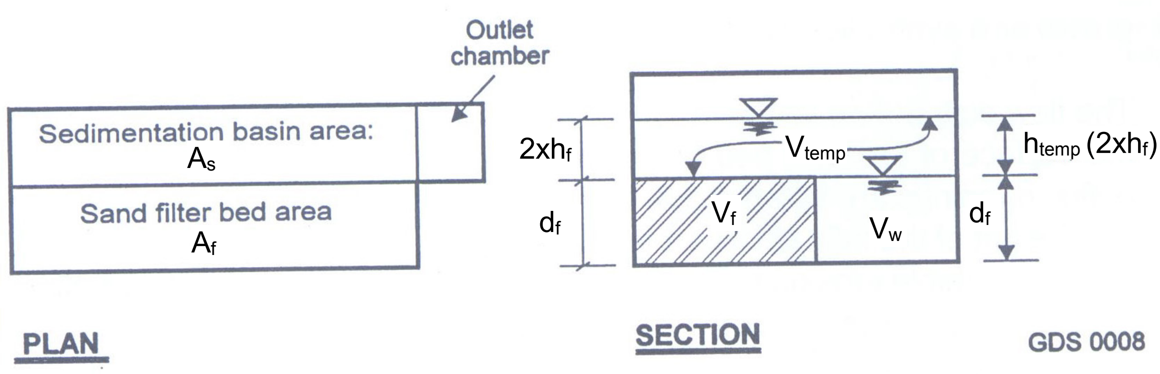


Figure 16.3 Perimeter Sand Filter Volumes
(Source: Claytor and Schueler, 1996)

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 16.5 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.

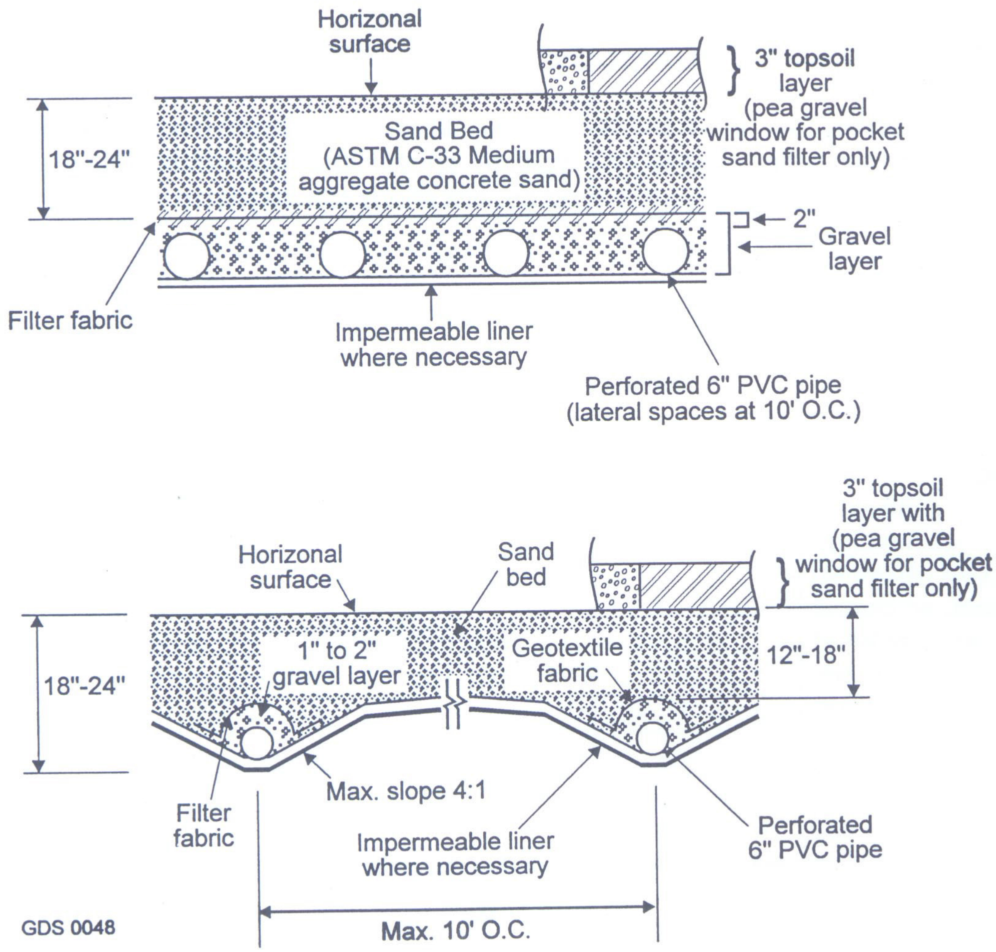


Figure 16.4 Typical Sand Filter Media Cross Sections
(Source: Claytor and Schueler, 1996)

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.

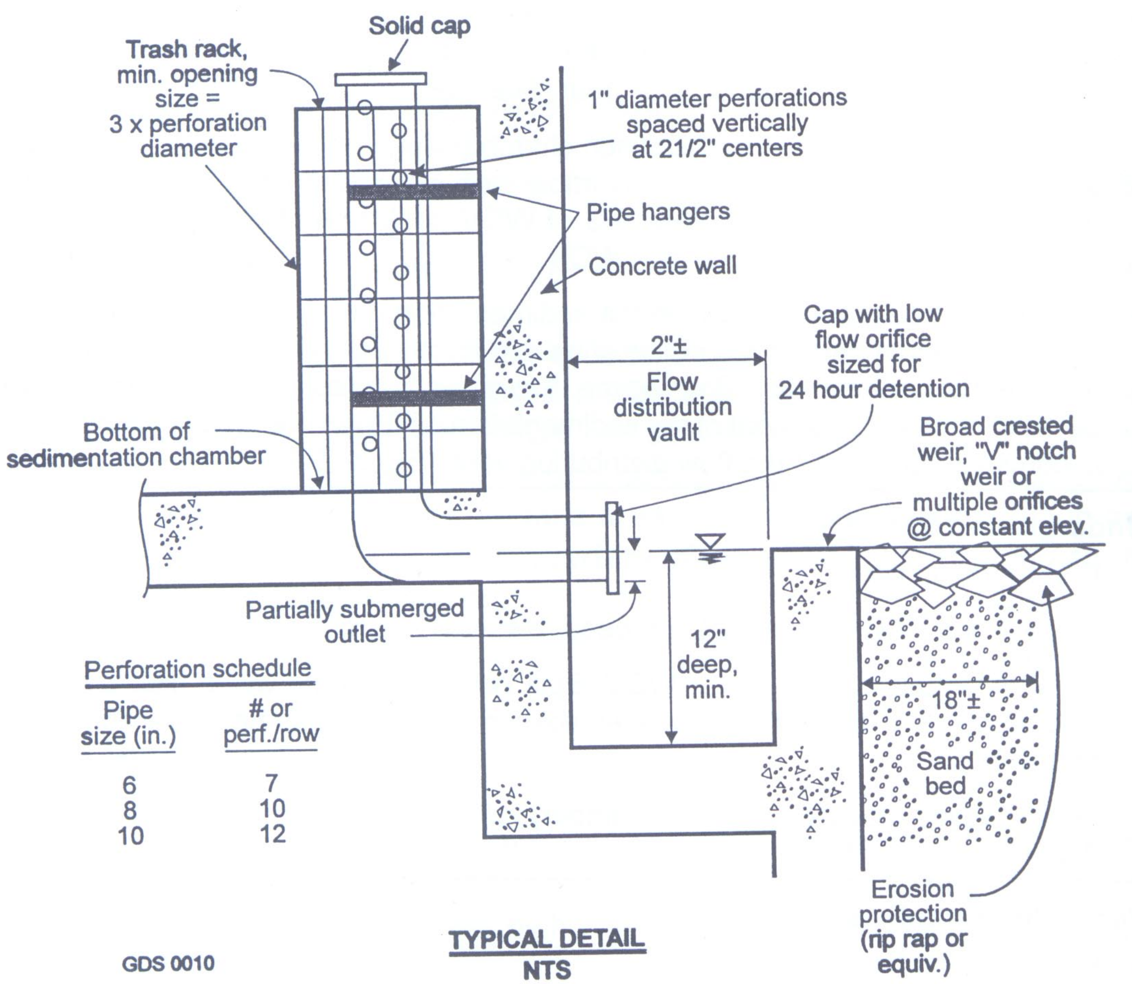


Figure 16.5 Surface Sand Filter Perforated Stand-Pipe
(Source: Claytor and Schueler, 1996)

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

16.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

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 Focus Areas are found in *Section 1.0 of the Planning Technical Manual*.

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 *Sections 16.4 and16.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 16.5* (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 *Section 1.0 of the Water Quality Technical Manual*).

(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, Equation 2.1):

$$A_f = (WQ_v)\,(d_f) / [(k)\,(h_f + d_f)\,(t_f)]$$

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 *Section 16.5* (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:

$$A_s = -(Q_o/w) * \text{Ln}\,(1-E) \tag{16.1}$$

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:

$$V_{min} = 0.75\ WQ_v = V_s + V_f + V_{f\text{-}temp} \tag{16.2}$$

(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 *Section 16.5* (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 *Section 29.3*).

(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 *Section 29.3* for a Sand Filter Design Example

16.7 Inspection and Maintenance Requirements

Table 16.1 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.

16.8 Example Schematics

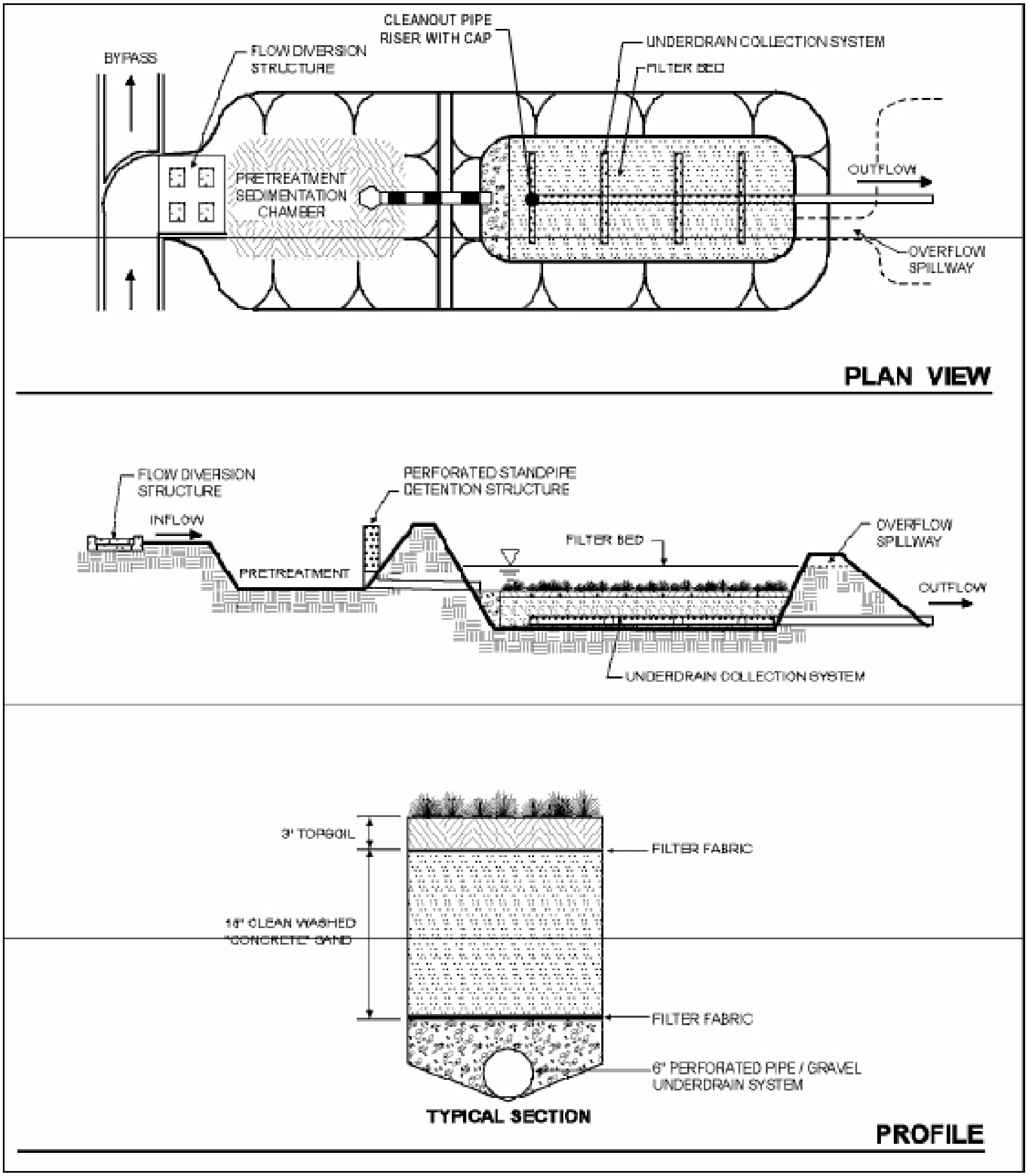


Figure 16.6 Schematic of Surface Sand Filter
(Source: Center for Watershed Protection)

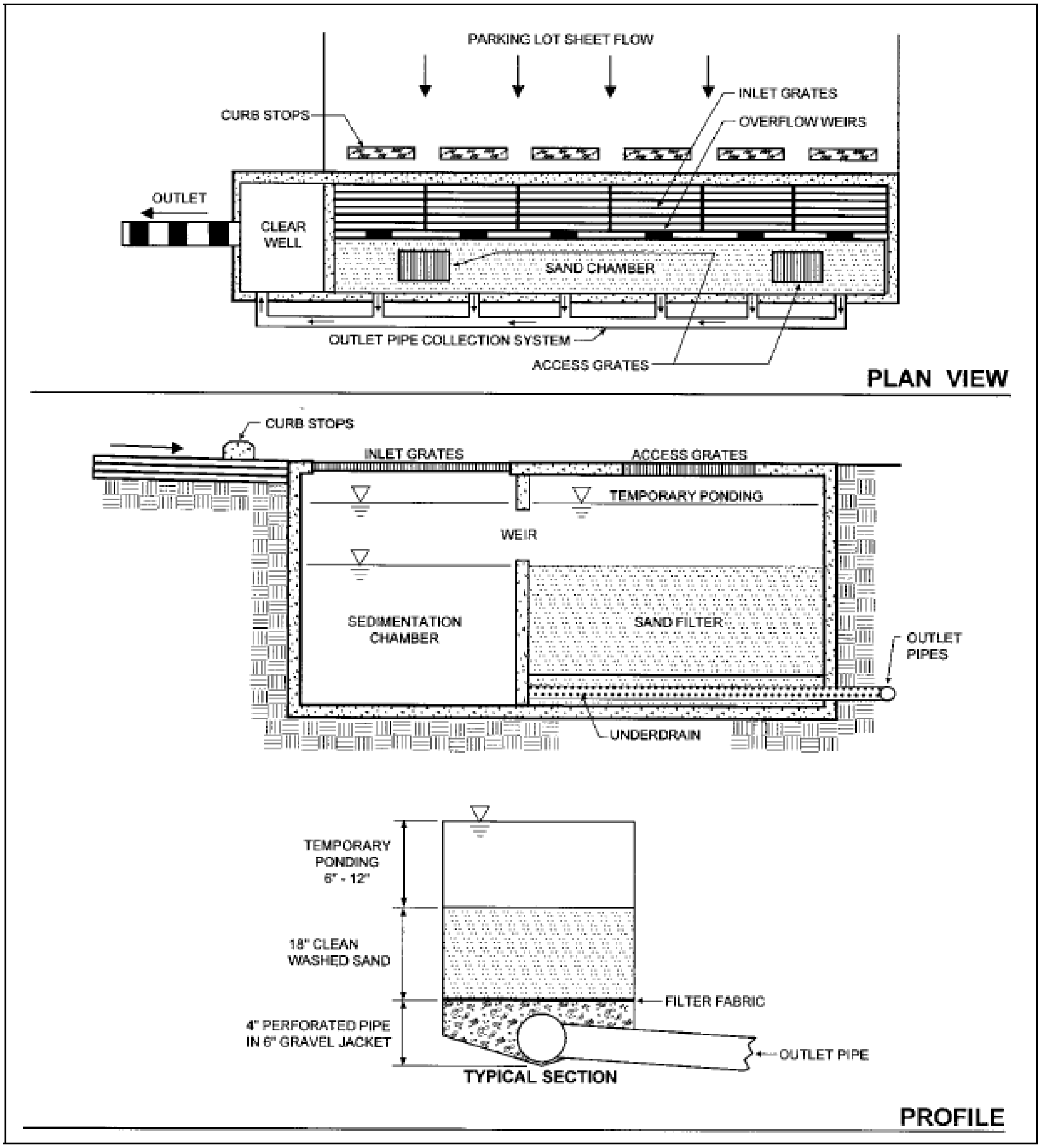


Figure 16.7 Schematic of Perimeter Sand Filter
(Source: Center for Watershed Protection)

16.9 Design Forms

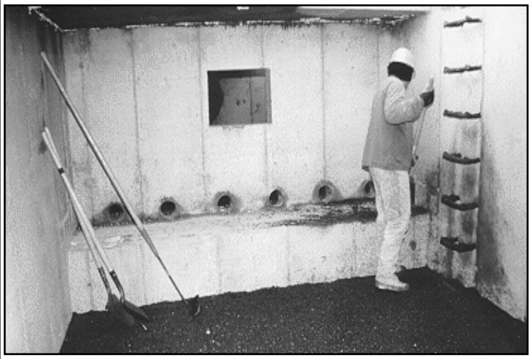
Design Procedure Form: Sand Filters

PRELIMINARY HYDROLOGIC	
1a. Compute WQ_v volume requirements	
Copute Runoff Coefficient, R_v	R_v = ____________
Compute WQ_v	WQ_v = ____________ acre-ft
1b. Compute SP_v	SP_v = ____________ acre-ft
Copute average release rate	release rate = ____________ cfs
Compute Q_p (100-year detention volume required)	Q_p = ____________ acre-ft
Compute (as necessary) Q_f	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	CN = ____________
Compute Time of Concentration t_c	t_c = ____________ hour
Compute Q_{wq}	Q_{wq} = ____________ cfs
5. Size flow diversion structure	
Low flow orifice - Orifice equation	A = ____________ ft^2
	diameter = ____________ in
Overflow weir - Weir equation	Length = ____________ ft
6. Size filtration bed chamber	
Compute area from Darcy's Law	A_f = ____________ ft^2
Using length to width (2:1) ratio	L = ____________ ft
	W = ____________ ft
7. Size seidmentation chamber	
Compute area from Camp-Hazen equation	A_f = ____________ ft^2
Given W from step 5, compute Length	L = ____________ ft
8. Compute V_{min}	V_{min} = ____________ ft^3
9. Compute volume within practice	
<u>Surface sand filter</u>	
Volume within filter bed	V_f = ____________ ft^3
Temporary storage above filter bed	$V_{s\text{-}temp}$ = ____________ ft^3
Sedimentation chamber (remaining volume)	V_s = ____________ ft^3
Height in sedimentation chamber	h_s = ____________ ft
Perforated stand pipe - Orifice equation	A = ____________ ft^2
	diameter = ____________ in
<u>Perimeter sand filter</u>	
Compute volume in filter bed	V_f = ____________ ft^3
Compute wet pool storage	V_w = ____________ ft^3
Compute temporary storage	V_{temp} = ____________ ft^3
	h_{temp} = ____________ ft
10. Compute overflow weir sizes	
Compute overflow - Orifice equation	Q = ____________ cfs
Weir from sedimentation chamber - Weir equation	Length = ____________ ft
Weir from filtration chamber - Weir equation	Length = ____________ ft

Notes:
____
____

17.0 Underground Sand Filter

Limited Application
Structural Stormwater 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 stormwater controls

STORMWATER 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

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

17.1 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 stormwater 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.

17.2 Pollutant Removal Capabilities

Underground sand filter pollutant removal rates are similar to those for surface and perimeter sand filters (see *Section 16.0*).

17.3 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:

$$\mathbf{V_w = A_s * 3 \text{ feet minimum}} \qquad \textbf{(17.1)}$$

- Consult the design criteria for the perimeter sand filter (see *Section 16.0*) for the rest of the underground filter sizing and design steps.

17.4 Inspection and Maintenance Requirements

Table 17.1 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 *Section 16.0*)

17.5 Example Schematic

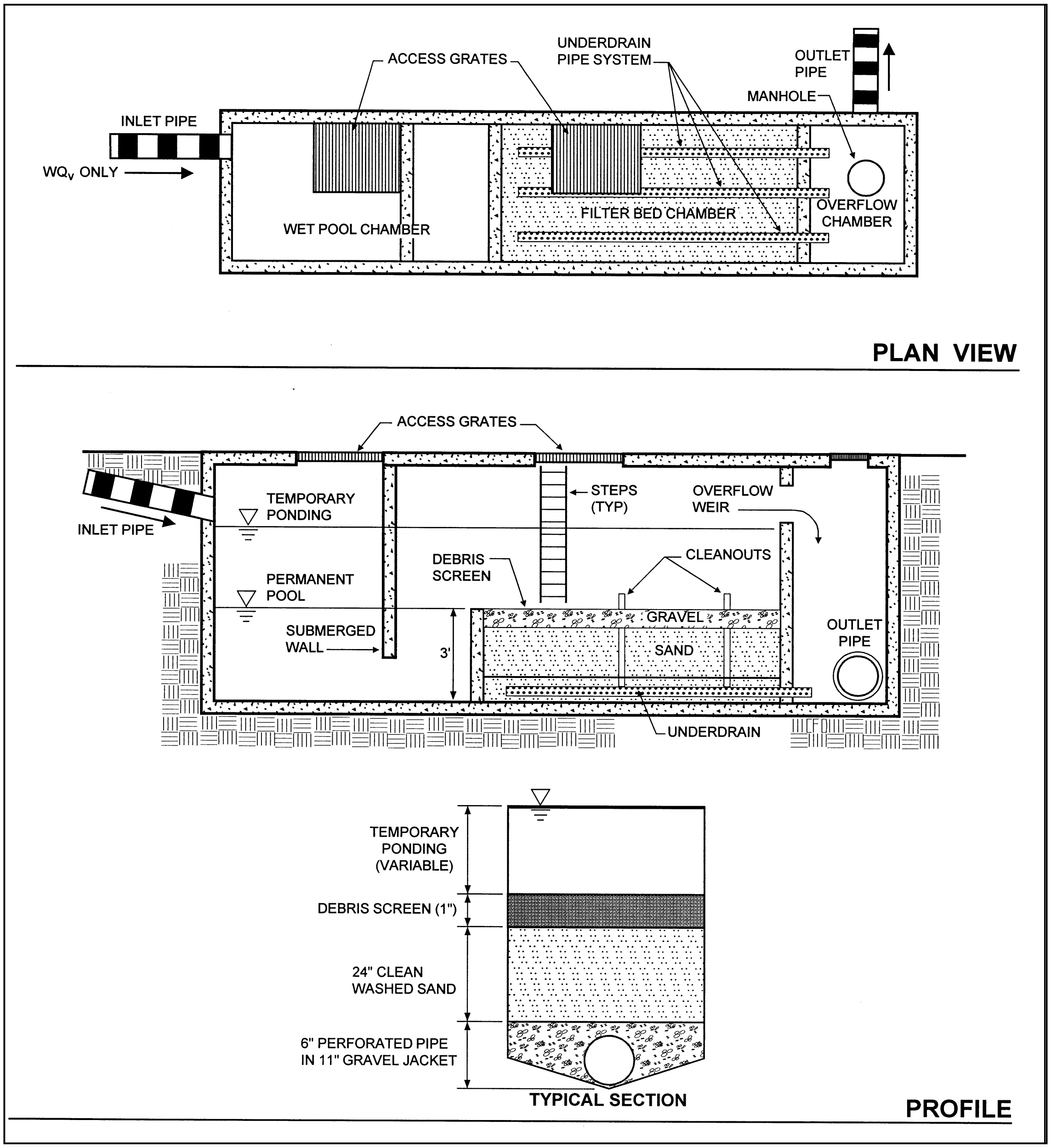
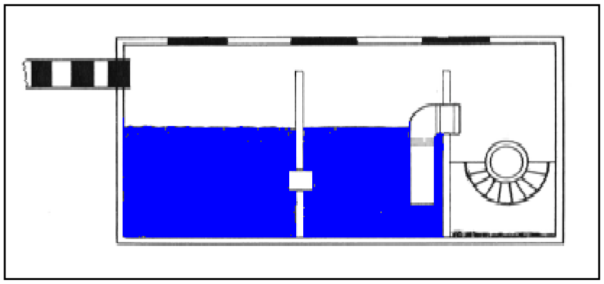


Figure 17.1 Schematic of Underground Sand Filter
(Source: Center for Watershed Protection)

18.0 Gravity (Oil Grit) Separator

Structural Stormwater Control



Description: Hydrodynamic separation device designed to remove settleable solids, oil and grease, debris, and floatables from stormwater 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

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

STORMWATER 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

18.1 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 stormwater 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.

18.2 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.

18.3 Design Criteria and Specifications

The use of gravity (oil-grit) separators should be limited to the following applications:

- Pretreatment for other structural stormwater 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 stormwater 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 volume for separated oil storage at the top of the chamber
- A volume for settleable solids accumulation at the bottom of the chamber
- A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater 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.

18.4 Inspection and Maintenance Requirements

Table 18.1 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 *Section 19.4*)

18.5 Example Schematic

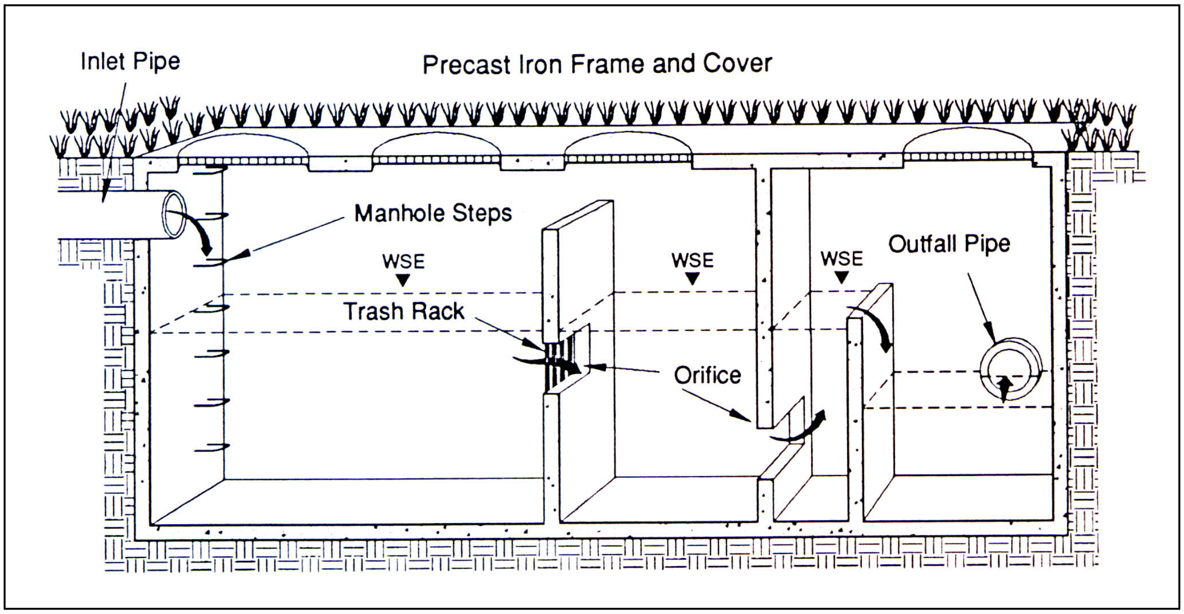
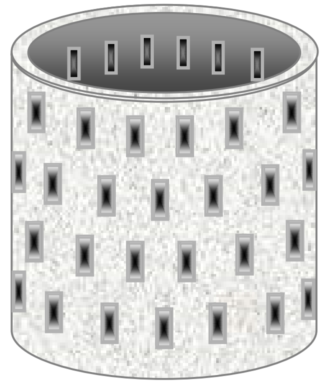


Figure 18.1 Schematic of an Example Gravity (Oil-Grit) Separator
(Source: NVRC, 1992[1])

19.0 Downspout Drywell

Structural Stormwater Control

Description: Drywells are essentially perforated manholes, but they can be manufactured in various sizes. Located underground, they allow stormwater 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 stormwater 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

STORMWATER 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

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

19.1 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 stormwater 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.

19.2 Pollutant Removal Capabilities

Pollutant removal is similar to infiltration trenches (see *Section 20.0*), 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%

19.3 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.

19.4 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 19.1 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

19.5 Example Schematic

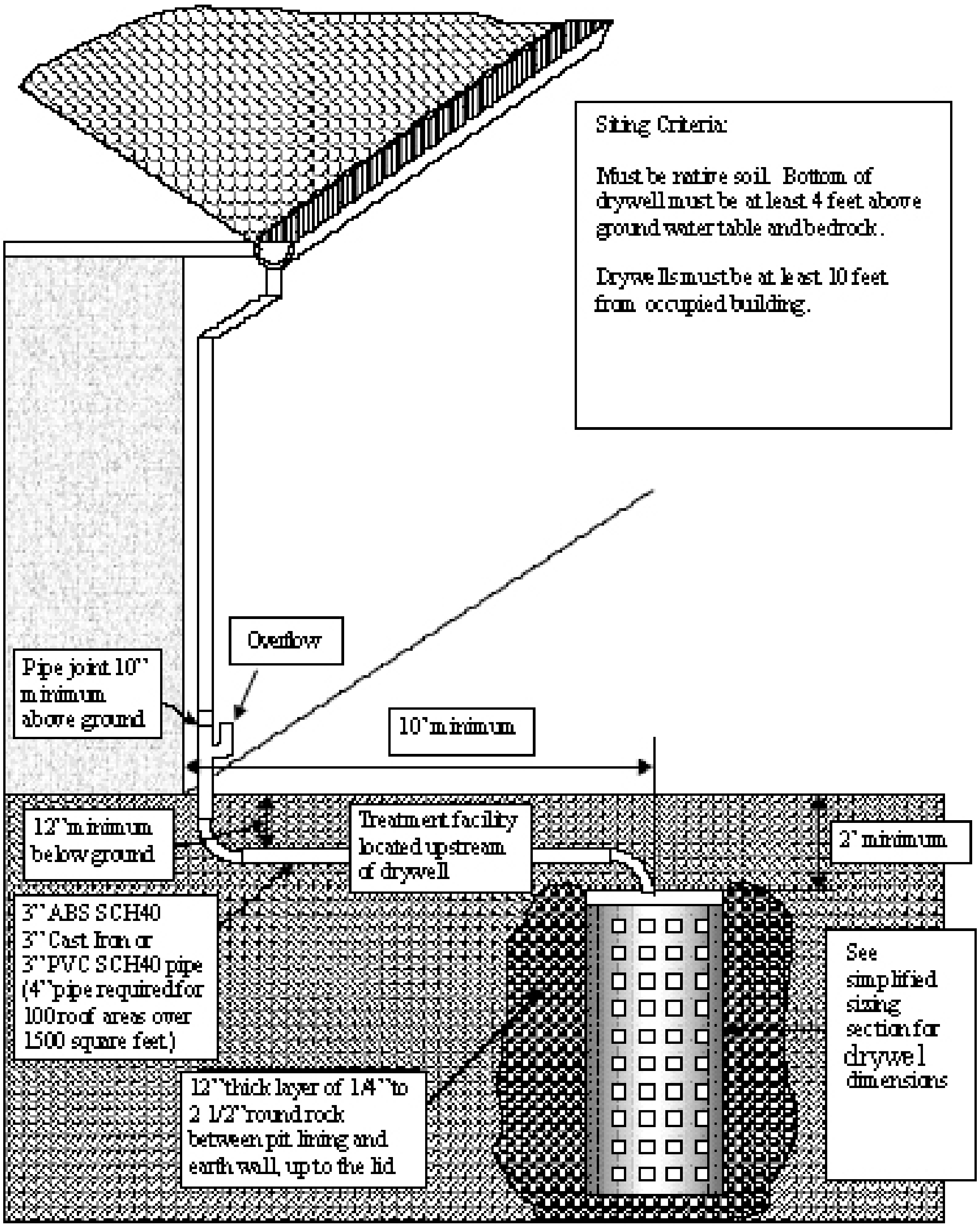


Figure 19.1 Schematic of Drywell System
(Source: City of Portland, Oregon)

20.0 Infiltration Trench

Structural Stormwater Control

Description: Excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater 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 (DRY SWALE)

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

STORMWATER 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

20.1 General Description

Infiltration trenches are excavations typically filled with stone to create an underground reservoir for stormwater runoff (see Figure 20.1). 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.

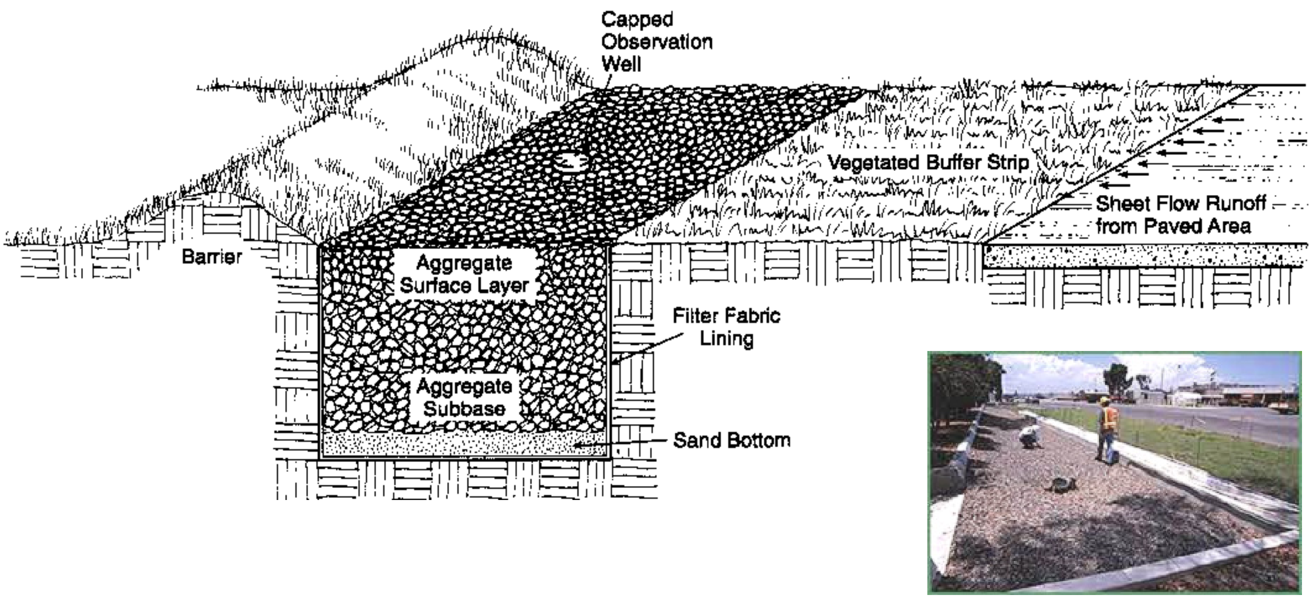


Figure 20.1 Infiltration Trench Example

20.2 Stormwater Management Suitability

Infiltration trenches are designed primarily for stormwater quality, i.e. the removal of stormwater 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.

<u>Water Quality Protection</u>

Using the natural filtering properties of soil, infiltration trenches can remove a wide variety of pollutants from stormwater 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 20.3 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 1.0).

20.3 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 Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

20.4 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 stormwater 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 stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO

Physical Feasibility - Physical Constraints at Project Site

- <u>Drainage Area</u> – 5 acres maximum
- <u>Space Required</u> – Will vary depending on the depth of the facility
- <u>Site Slope</u> – No more than 6% slope (for pre-construction facility footprint)
- <u>Minimum Head</u> – Elevation difference needed at a site from the inflow to the outflow: 1 foot
- <u>Minimum Depth to Water Table</u> – 4 feet recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table
- <u>Soils</u> – Infiltration rate greater than 0.5 inches per hour required (typically hydrologic group "A", some group "B" soils)

Other Constraints / Considerations

- <u>Aquifer Protection</u> – No hotspot runoff allowed; meet setback requirements in design criteria

20.5 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 <u>0.5 inches per hour or greater</u>, 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. Stormwater 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 20.2 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 20.1.

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 20.3 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

20.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Protection Volume (WQ_v), Streambank Protection Volume (SP_v), and the Flood mitigation storm (Q_f).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual*.

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 *Sections 20.4 and 20.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 20.5* (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 *Section 1.5 of the Hydrology Technical Manual*).

(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:

$$A = \frac{WQ_v}{(nd + kT/12)} \quad \textbf{(20.1)}$$

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 *Section 20.5* (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 *Section 20.5* (D) (Pretreatment / Inlets) for more details.

Step 8 Design spillway(s)

Adequate stormwater outfalls should be provided for the overflow exceeding the capacity of the trench, ensuring nonerosive velocities on the down-slope.

See *Section 29.5* for an Infiltration Trench Design Example

20.7 Inspection and Maintenance Requirements

Table 20.1 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

Table 20.1 Typical Maintenance Activities for Infiltration Trenches	
Activity	**Schedule**
• 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.

20.8 Example Schematics

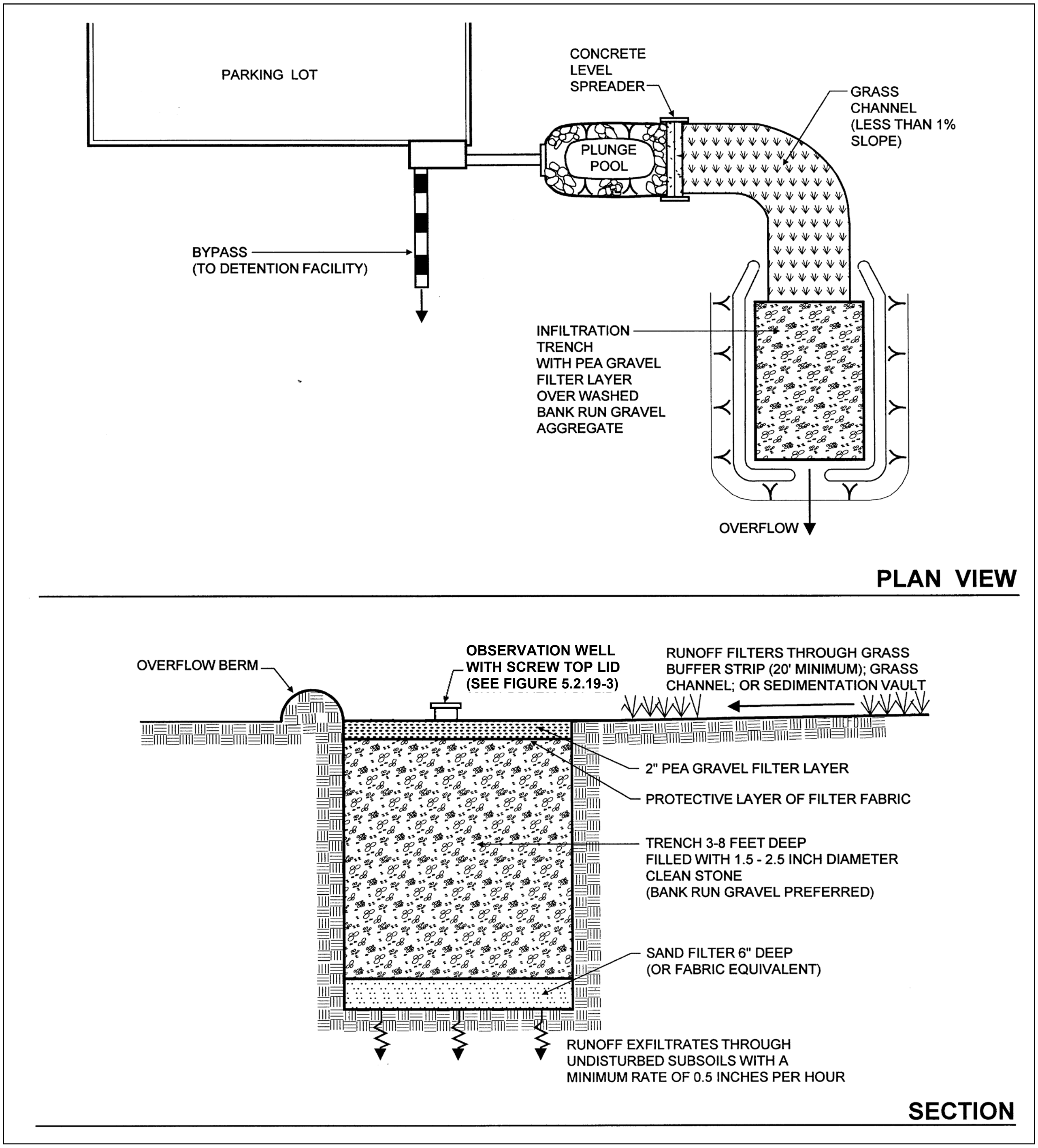


Figure 20.2 Schematic of Infiltration Trench
(Source: Center for Watershed Protection)

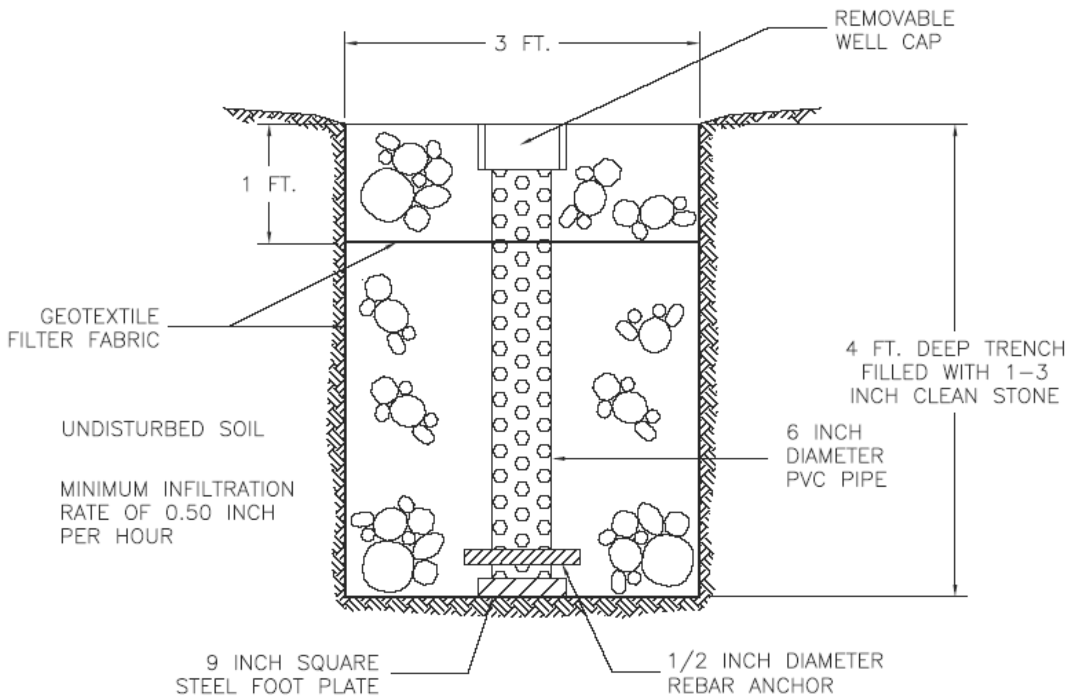


Figure 20.3 Observation Well Detail

- 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.

20.9 Design Forms

Design Procedure Form: Infiltration Trench

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = __________
Compute WQv	WQ_v = __________ acre-ft
1b. Compute $\mathbf{SP_v}$	$\mathbf{SP_v}$ = __________ acre-ft
Compute average release rate	release rate = __________ cfs
Compute Q_p (100-year detention volume required)	Q_p = __________ acre-ft
Compute (as necessary) Q_f	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	CN = __________
Compute Time of Concentration t_c	t_c = __________ hour
Compute Q_{wq}	Q_{wq} = __________ cfs
5. Size infiltration trench	Area = __________ ft^2
Width must be less than 25 ft	Width = __________ ft
	Length = __________ ft
6. Size the flow diversion structures	
Low flow orifice from orifice equation	
$Q = CA(2gh)^{0.5}$	A = __________ ft^2
	diam. = __________ inch
Overflow weir from weir equation	
$Q = CLH^{3/2}$	Length = __________ ft
7. Pretreatment volume (for offine designs)	
$Vol_{pre} = 0.25(WQ_v)$	Vol_{pre} = __________ ft^3
8. Design spillway(s)	

Notes: ____

21.0 Soakage Trench

Structural Stormwater 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 stormwater 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 stormwater 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

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
S	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

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

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

21.1 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 stormwater disposal are considered Class V injection devices by the EPA and fall under the Texas UIC program.

21.2 Pollutant Removal Capabilities

Pollutant removal is similar to infiltration trenches (see *Section 16.0*), 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%

21.3 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 21.1. 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.

21.4 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 21.1 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

21.5 Example Schematics

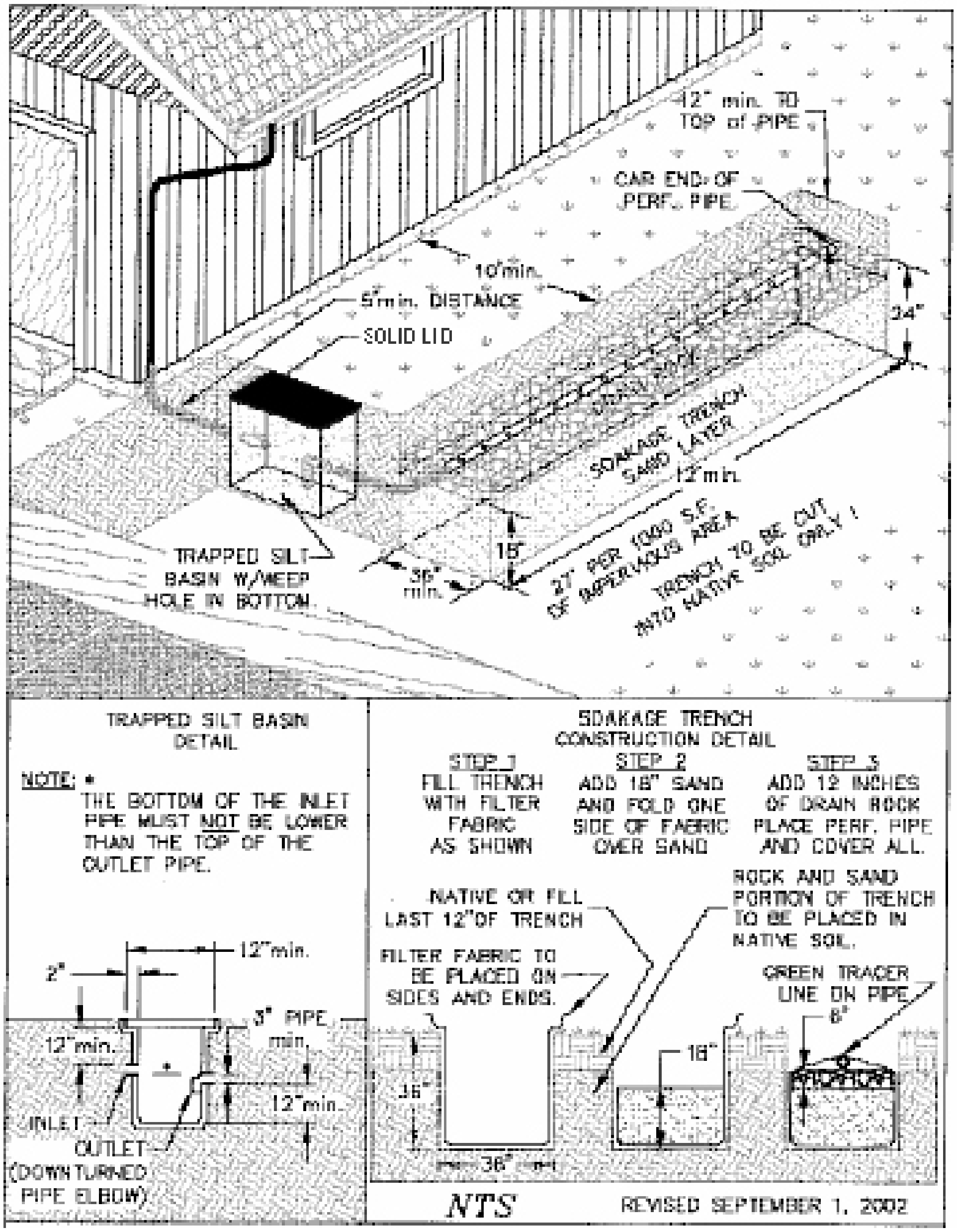


Figure 21.1 Schematic of a Soakage Trench
(Source: City of Portland, Oregon)

Stormwater Control

22.0 Stormwater Ponds

Description: Constructed stormwater 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

STORMWATER 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	**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

22.1 General Description

Stormwater ponds (also referred to as *retention ponds*, *wet ponds*, *or wet extended detention ponds*) are constructed stormwater 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 stormwater 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 stormwater 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.

Stormwater ponds are among the most cost-effective and widely used stormwater 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 stormwater 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 stormwater 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 stormwater basins constructed with a permanent (dead storage) pool of water equal to the water quality volume. Stormwater 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 22.1 shows a number of examples of stormwater pond variants. *Section 22.8* 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 stormwater 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 *Section 10.0* on Dry Detention Basins).

Wet Pond | Wet ED Pond

Micropool ED Pond | Wet ED Pond

Figure 22.1 Stormwater Pond Examples

22.2 Stormwater Management Suitability

Stormwater ponds are designed to control both stormwater quantity and quality. Thus, a stormwater pond can be used to address all of the *integrated stormwater sizing criteria* for a given drainage area.

<u>Water Quality</u>

Ponds treat incoming stormwater runoff by physical, biological, and chemical processes. The primary removal mechanism is gravitational settling of particulates, organic matter, metals, bacteria, and organics as stormwater 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 stormwater contaminants such as hydrocarbons.

Section 22.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

<u>Streambank Protection</u>

A portion of the storage volume above the permanent pool in a stormwater 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).

<u>On-Site Flood Control</u>

A stormwater 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, stormwater ponds can also be used to provide detention to control the flood mitigation storm peak flow downstream. Where this is not required, the pond structure is designed to safely pass extreme storm flows.

22.3 Pollutant Removal Capabilities

All of the stormwater 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 stormwater ponds, 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

22.4 Application and Site Feasibility Criteria

Stormwater 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 stormwater pond for meeting stormwater 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 Stormwater 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 stormwater pond outflows on downstream local aquatic habitats.

22.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a stormwater 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

Stormwater 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 *Section 4.0 of the Hydrology Technical Manual* for details). Ensure that an appropriate anti-clogging device is provided for the pond outlet.

A stormwater 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.

Stormwater ponds should not be located on steep (>15%) or unstable slopes.

Stormwater 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 stormwater 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 stormwater 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 stormwater 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 *iSWM Program Guidance – Federal, State and Regional Initiatives*)

Additional pond design features include an emergency spillway, maintenance access, safety bench, pond buffer, and appropriate native landscaping.

Figures 22.4 thru 22.7 in *Section 22.8* 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 22.2). 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 22.2).

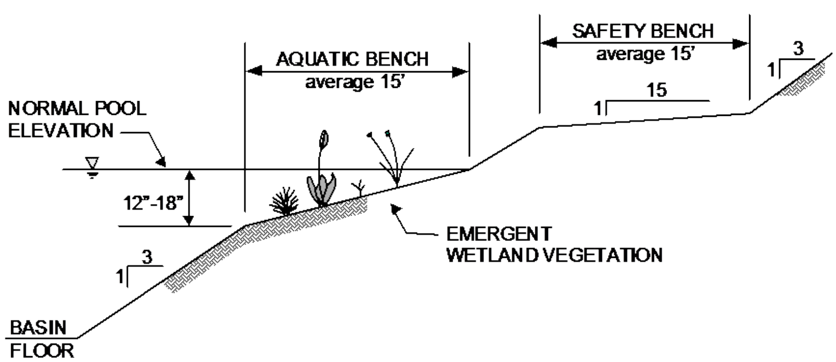


Figure 22.2 Typical Stormwater Pond Geometry Criteria

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 stormwater 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 stormwater 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 22.3). The riser should be located within the embankment for maintenance access, safety and aesthetics.

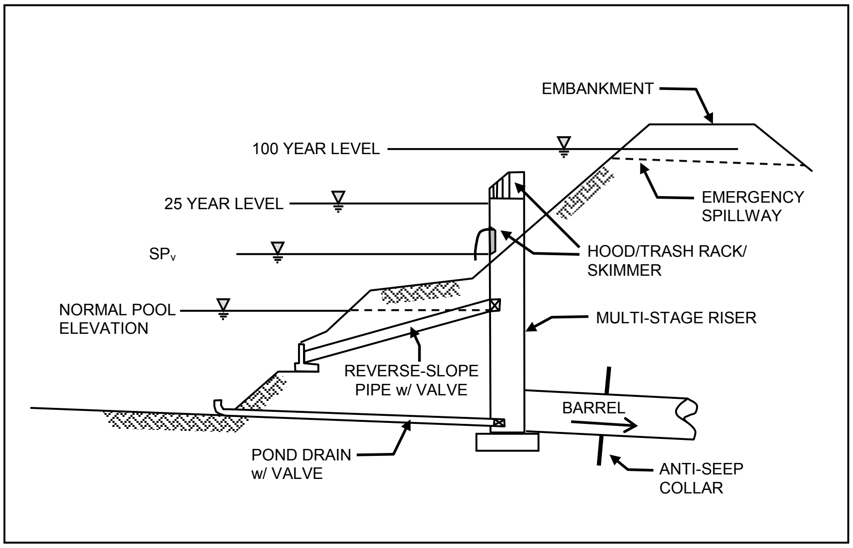


Figure 22.3 Typical Pond Outlet Structure

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 *iSWM Program Guidance – Dams and Reservoirs in Texas*.

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 4.0 of the Hydraulics Technical Manual* 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 *Section 22.6* as well as *Sections 2.0 and 2.2 of the Hydraulics Technical Manual* for additional information and specifications on pond routing and outlet works.

F Emergency Spillway

An emergency spillway is to be included in the stormwater 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 *iSWM Program Guidance – Dams and Reservoirs in Texas).*

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 stormwater 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 the *Landscape Technical Manual.*
- 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 stormwater 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 stormwater pond locations is strongly encouraged.

J Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

- <u>Low Relief</u> – 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 the *Landscape Technical Manual*); provide 48-hour extended detention for maximum coliform dieoff.

Dams

Dam construction for stormwater ponds can take a variety of forms. Large dams that are over six feet in height are regulated by the State of Texas (See *iSWM Program Guidance – Dams and Reservoirs in Texas*). 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 stormwater 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.

22.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

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 Focus Areas are found in *Section 1.0 of the Planning Technical Manual.*

Step 2 Determine if the development site and conditions are appropriate for the use of a stormwater pond

Consider the Application and Site Feasibility Criteria in *Sections 22.4 and 22.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 22.5 (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 *Section 22.5* (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 flood mitigation stormwater surface elevation, set top of embankment elevation, and analyze safe passage of the flood mitigation storm.

At final design, provide safe passage for the flood mitigation storm event.

Step 10 Investigate potential pond hazard classification

The design and construction of stormwater management ponds are required to follow the latest version of the State of Texas Administrative Code for and reservoirs (see *iSWM Program Guidance - Dams and Reservoirs in Texas*).

Step 11 Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See *Section 22.5* (D) through (H) for more details.

Step 12 Prepare Vegetation and Landscaping Plan

A landscaping plan for a stormwater pond and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See *Section 22.5* (I) (Landscaping) and the *Landscape Technical Manual* for more details.

See *Section 29.2* for a Stormwater Pond Design Example

22.7 Inspection and Maintenance Requirements

Table 22.1 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 stormwater 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 stormwater pond.

Regular inspection and maintenance is critical to the effective operation of stormwater 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.

22.8 Example Schematics

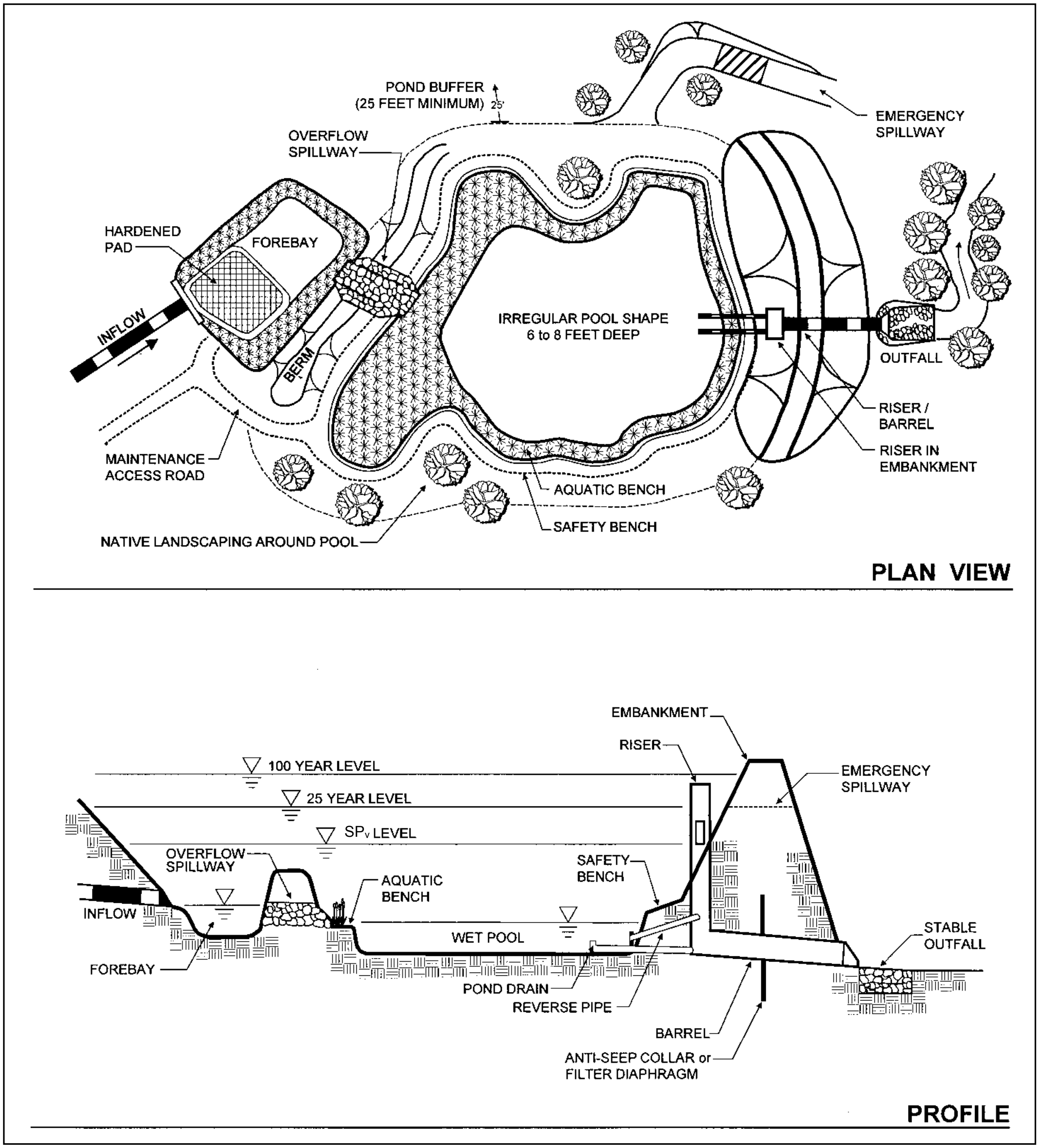


Figure 22.4 Schematic of Wet Pond
(Source: Center for Watershed Protection)

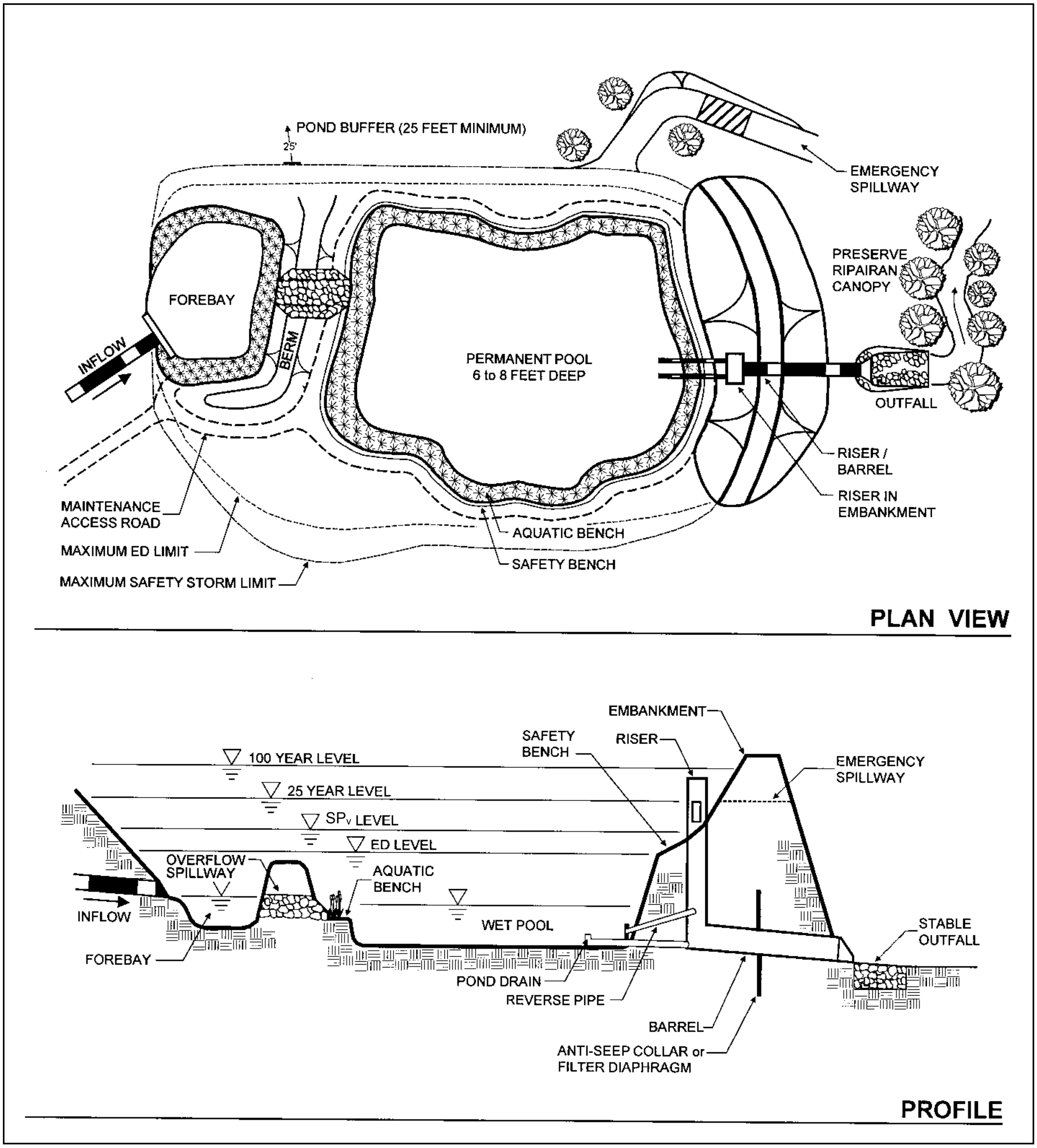


Figure 22.5 Schematic of Wet Extended Detention Pond
(Source: Center for Watershed Protection)

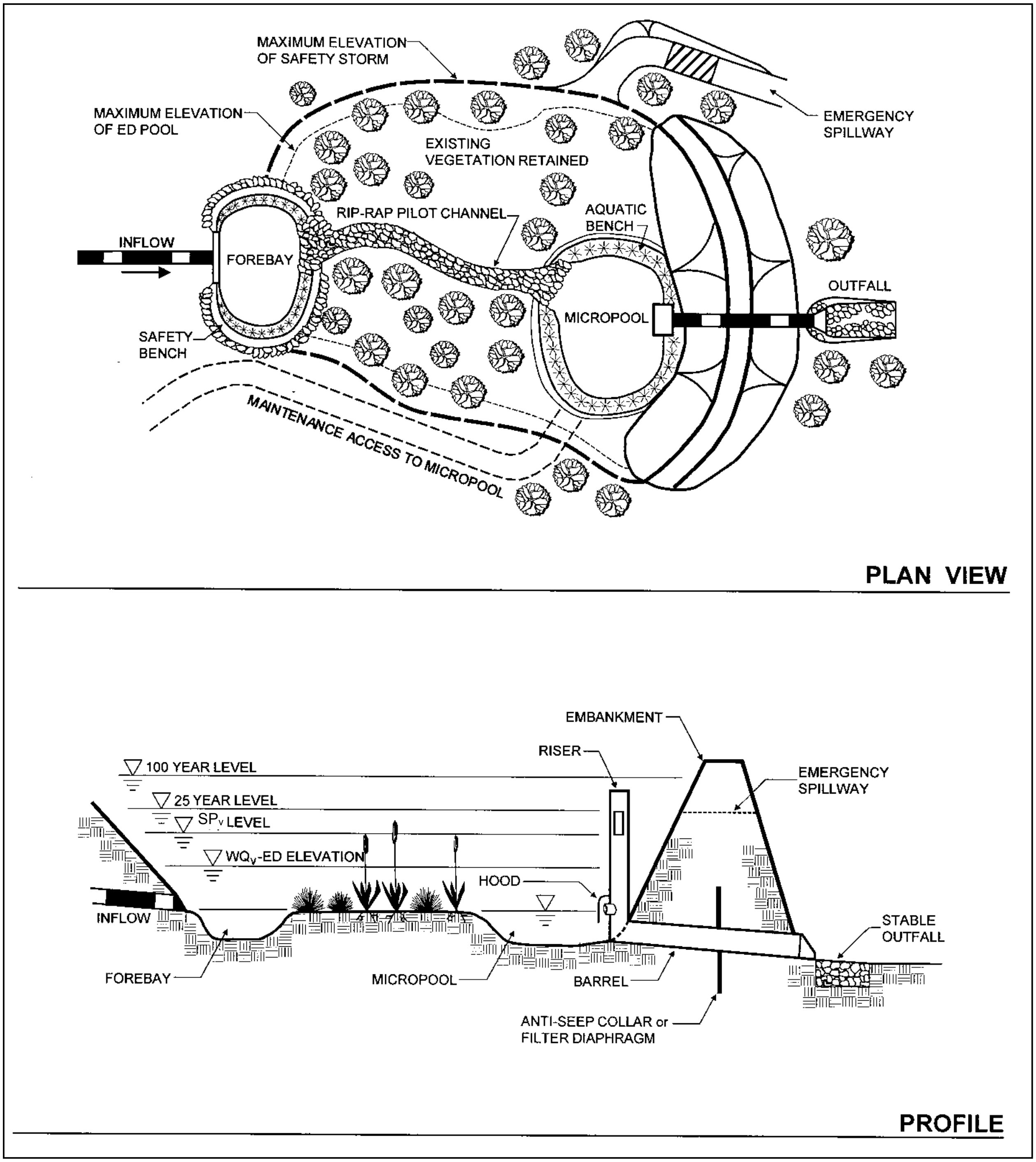


Figure 22.6 Schematic of Micropool Extended Detention Pond
(Source: Center for Watershed Protection)

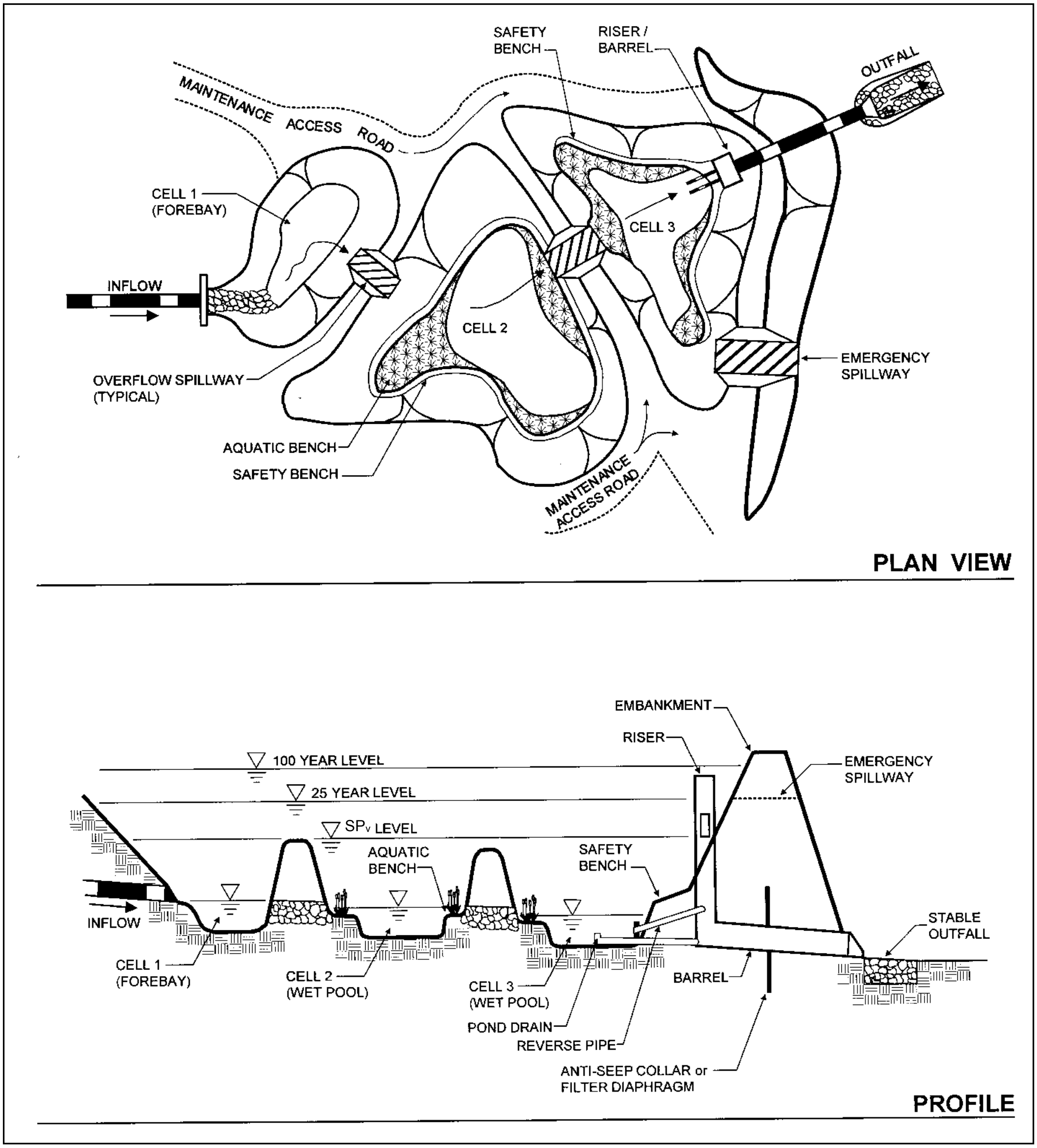


Figure 22.7 Schematic of Multiple Pond System
(Source: Center for Watershed Protection)

22.9 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	release rate = ________ cfs
Average ED release rate (if applicable)	h = ________ ft
Average head, h = (ED elev. - Permanent pool elev.) / 2	
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 = ________ cfs
Release rate =	h = ________ ft-NGVD
Aerage head, h = SP_v elev. - Permanent pool elev.) / 2	A = ________ ft^2
Area of orifice from orifice equatin	diameter = ________ in
$Q = CA(2gh)^{0.5}$	factor = ________ $(h)^{0.5}$
Discharge equation $Q = (h)^{0.5}$	
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 Orif.	High Storage Weir	Inlet	Pipe	Spillway	Storage
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**

23.0 Green Roof

Structural Stormwater 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.

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
S	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**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

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

23.1 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 stormwater 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 stormwater 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.

23.2 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 23.1. 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.

23.3 Inspection and Maintenance Requirements

Table 23.1 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

23.4 Example Schematic

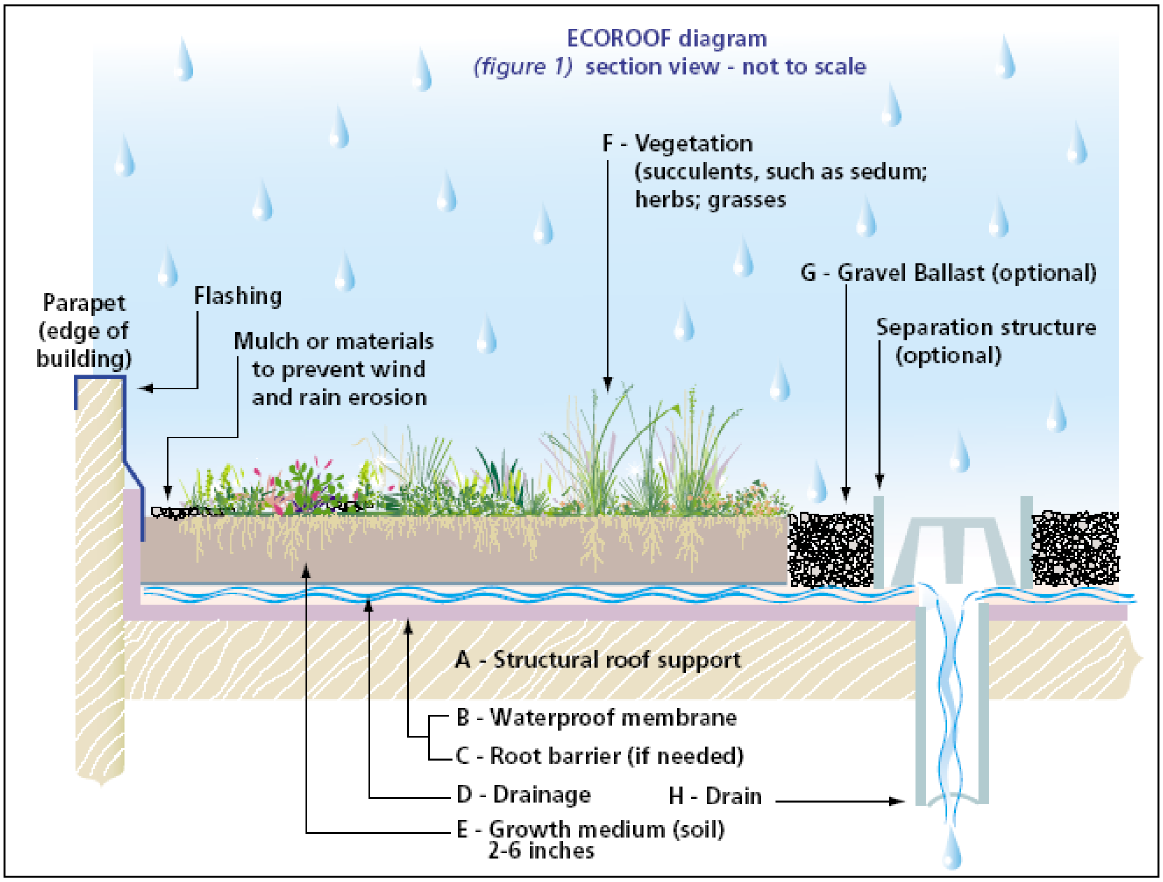
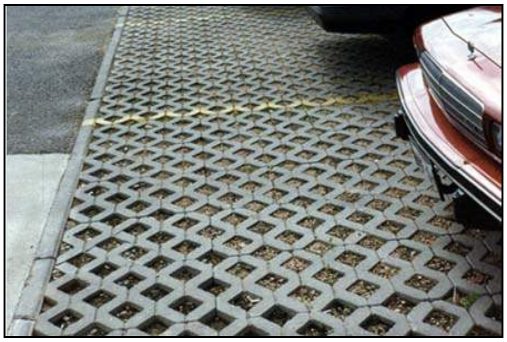


Figure 23.1 Green Roof Cross Section (from City of Portland, Oregon)

24.0 Modular Porous Paver Systems

Structural Stormwater 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

STORMWATER 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

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

24.1 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 stormwater runoff. Porous paver systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater 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 24.1).

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.

24.2 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater 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%

24.3 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:

$$\mathbf{d = V / A * n} \qquad \textbf{(24.1)}$$

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 1.0 of the Planning Technical Manual*.

24.4 Inspection and Maintenance Requirements

Table 24.1 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

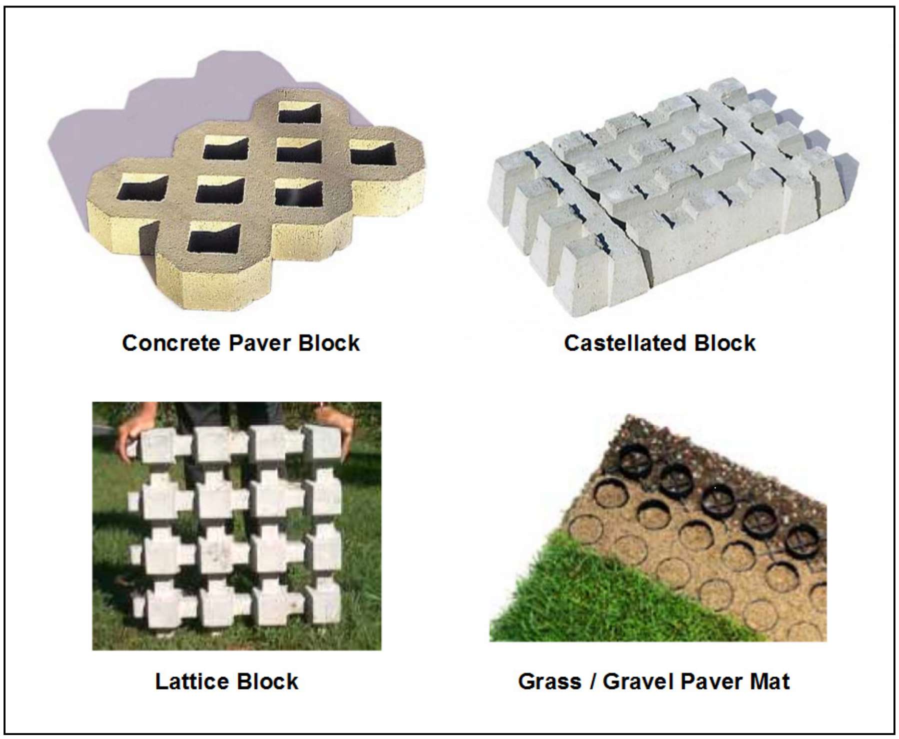


Figure 24.1 Examples of Modular Porous Pavers

24.5 Example Schematics

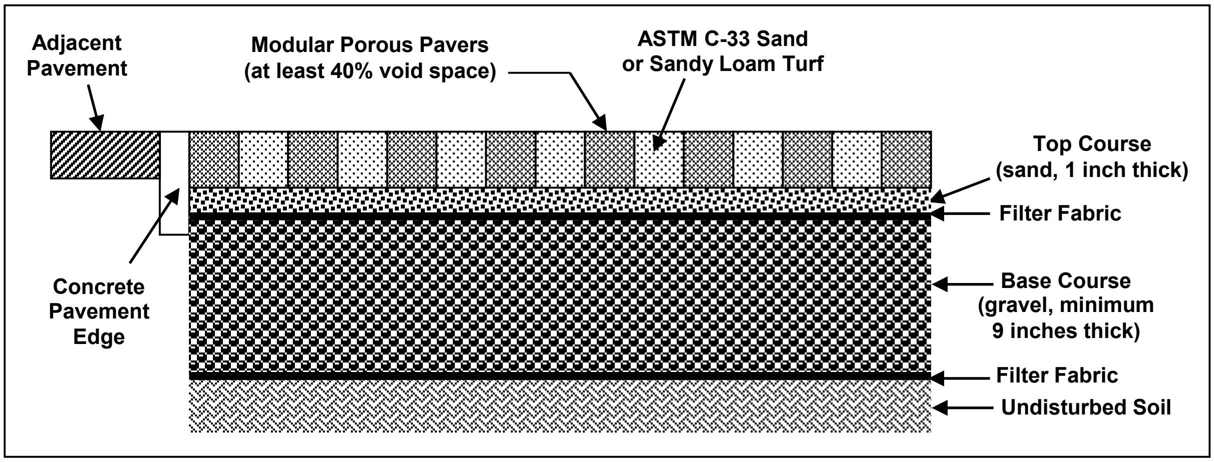


Figure 24.2 Modular Porous Paver System Section

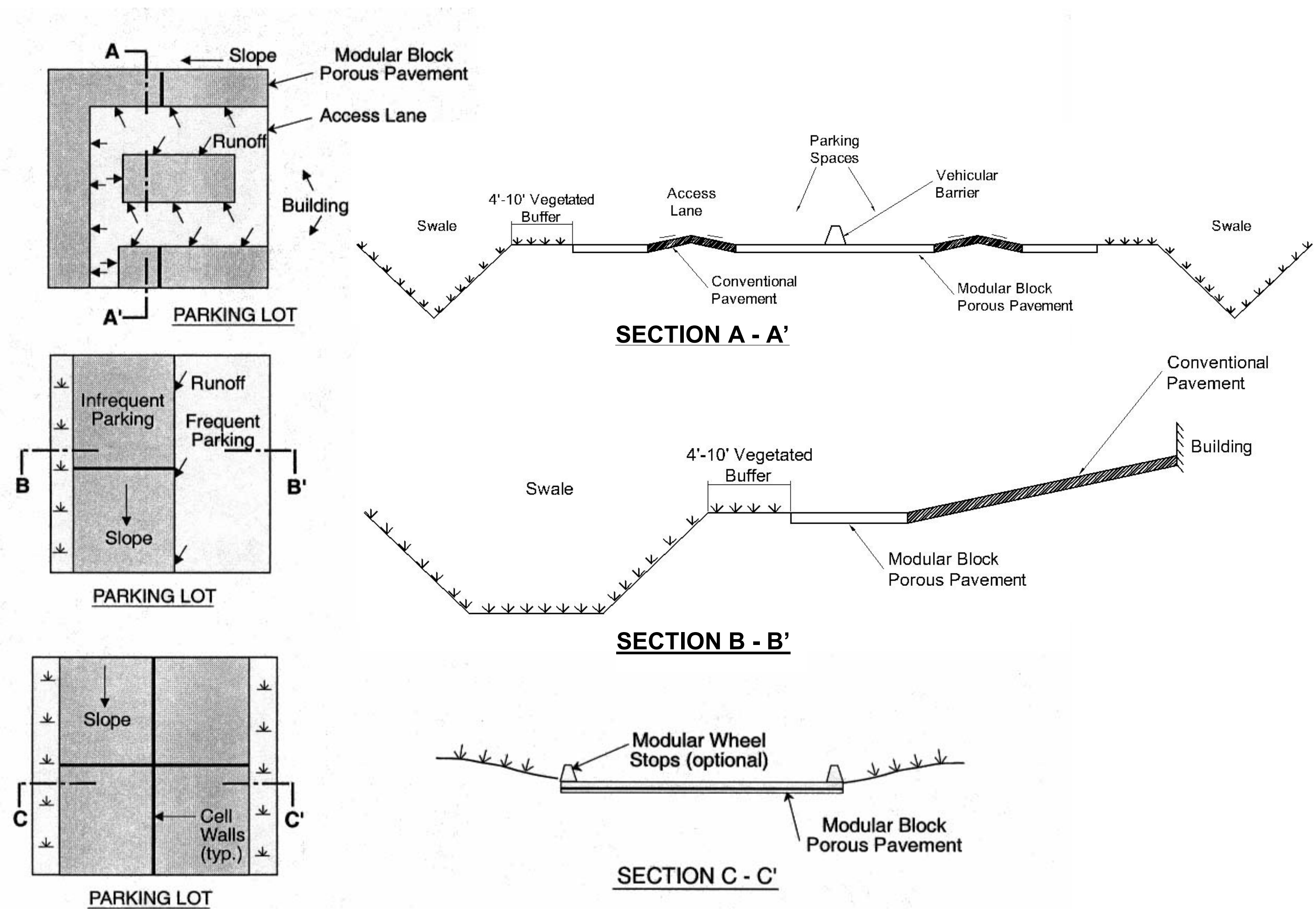
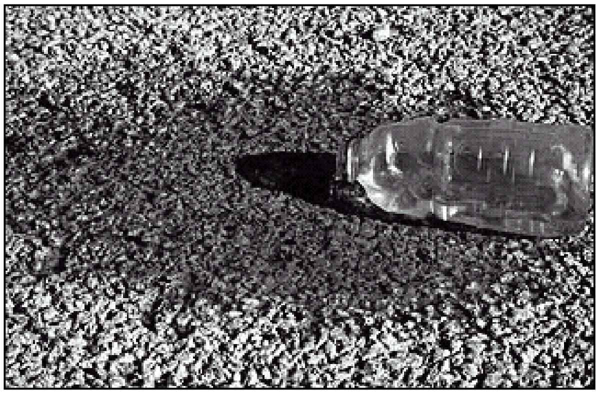


Figure 24.3 Typical Modular Porous Paver System Applications
(Source: UDFCD, 1999)

Figure 24.4 Examples of Porous Paver Surfaces
(Sources: Invisible Structures, Inc.; EP Henry Corp.)

25.0 Porous Concrete

Limited Application
Structural Stormwater 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

STORMWATER 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

POLLUTANT REMOVAL

NA	**Total Suspended Solids**
50/65%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
60%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No	**Pathogens** - Coliform, Streptococci, E.Coli removal

25.1 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 stormwater quality, i.e. the removal of stormwater 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 stormwater 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.

25.2 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater 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).

25.3 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 4.0 and 13.0*)

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 1.0 of the Planning Technical Manual.*

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 two hours 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 25.1. 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:

- *Porous Concrete Layer* – 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.
- *Top Filter Layer* – 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.
- *Reservoir Layer* – 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).

25.4 Inspection and Maintenance Requirements

Table 25.1 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

Table 25.1 Typical Maintenance Activities for Porous Concrete Systems	
Activity	**Schedule**
• 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.

25.5 Example Schematics

Figure 25.1 Porous Concrete System Section
(Modified From: LAC 2000)

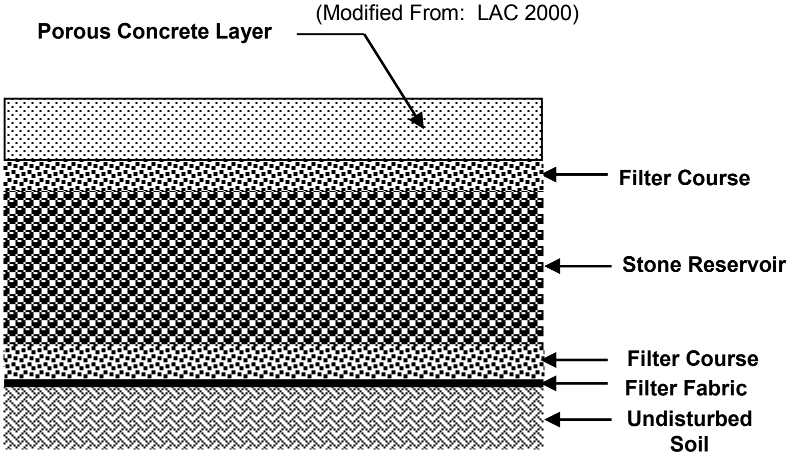


Figure 25.2 Porous Concrete System Installation

Figure 25.3 Typical Porous Concrete System Applications
(Photos by Bruce Ferguson, Don Wade)

25.6 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 Equation 20.1:

$$A = WQ_v/(n_g d_g + kT/12 + n_p d_p)$$

$$= 7{,}759/(0.32*3 + 1.02*2/12 + 0.18 * 3/12)$$

$$= 6{,}604 \text{ square feet}$$

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.

26.0 Proprietary Structural Controls

Limited Application
Structural Stormwater Control

Description: Manufactured structural control systems available from commercial vendors designed to treat stormwater runoff and/or provide water quantity control

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Independent performance data must be available to prove a demonstrated capability of meeting stormwater 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

STORMWATER 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 section is being included in order to provide communities with a rationale for approving the use of a proprietary system or practice in their jurisdictions.

26.1 General Description

There are many types of commercially-available proprietary stormwater 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.

26.2 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 stormwater 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 stormwater controls.

27.0 Rain Harvesting (Tanks/Barrels)

Stormwater 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)

STORMWATER 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

27.1 General Description

Rain harvesting (also referred to as *rain pails* and *rain savers*) are used as a water conservation practice and a stormwater 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.

27.2 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 27.1 $V = A^2 \times R \times 0.90 \times 7.5$ *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. $V = 212 \text{ ft.}^2 \times 0.125 \text{ ft.} \times 0.90 \times 7.5 \text{ gallons/ft.}^3 = 179 \text{ gallons}$

- 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.

27.3 Example Schematics

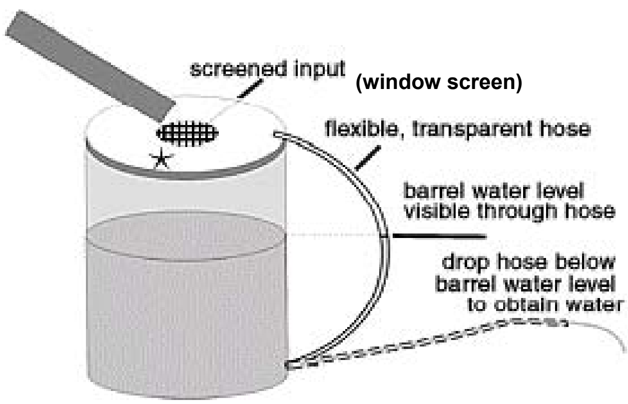


Figure 27.1 Simple Rain Barrel Design (from Maryland DNR Green Building Program)

Figure 27.2 Rain barrels in series

28.0 Stormwater Wetlands

Stormwater Control

Description: Constructed wetland systems used for stormwater 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

STORMWATER 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

28.1 General Description

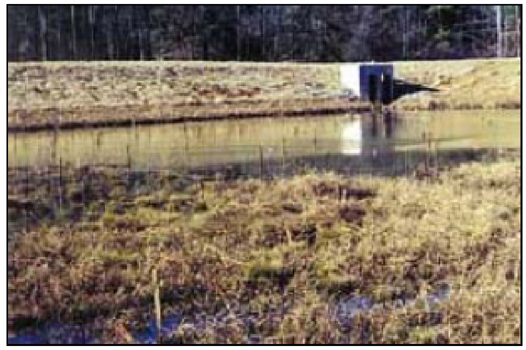
Stormwater wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes. As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

Wetlands are among the most effective stormwater practices in terms of pollutant removal and also offer aesthetic value and wildlife habitat. Constructed stormwater wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for the purpose of treating stormwater runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, stormwater wetlands require a continuous base flow or a high water table to support aquatic vegetation.

There are several design variations of the stormwater 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 28.1 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 stormwater 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 stormwater 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. Stormwater 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 stormwater 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 28.1 Stormwater Wetland Examples

28.2 Stormwater Management Suitability

Similar to stormwater ponds, stormwater wetlands are designed to control both stormwater quantity and quality. Thus, a stormwater wetland can be used to address all of the integrated stormwater sizing criteria for a given drainage area.

Water Quality

Pollutants are removed from stormwater 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 stormwater wetland including chemical and biological decomposition and volatilization. *Section 28.3* 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 stormwater 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, stormwater wetlands can also be used to provide detention to control the flood mitigation storm peak flow. Where flood mitigation storm peak control is not required, a stormwater wetland must be designed to safely pass the flood mitigation storm flows.

28.3 Pollutant Removal Capabilities

All of the stormwater 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 stormwater wetlands, 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

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.

28.4 Application and Site Feasibility Criteria

Stormwater 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 stormwater wetland for meeting stormwater 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 Stormwater 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 stormwater 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*.

28.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a stormwater 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

Stormwater 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 stormwater wetland can withstand a 30-day drought at summer evaporation rates without completely drawing down (see *Section 4.0 of the Hydrology Technical Manual* 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.

Stormwater 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 stormwater 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 stormwater 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 stormwater pond facility (see *Section 22.0*, 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 28.3 through 28.6 in *Section 28.8* 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 stormwater wetland that must be observed for adequate pollutant removal, ease of maintenance, and improved safety. Table 28.1 provides the recommended physical specifications and geometry for the various stormwater wetland design variants.

Table 28.1 Recommended Design Criteria for Stormwater 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 stormwater 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 stormwater wetland surface area. Target allocations are found in Table 28.1. 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 stormwater 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 stormwater ponds (see *Section 22.0*).

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 *Section 4.0 of the Hydrology Technical Manual* 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 the *Landscape Technical Manual*.

E Pretreatment / Inlets

Sediment regulation is critical to sustain stormwater wetlands. A wetland facility should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater 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 stormwater 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 28.2). 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.

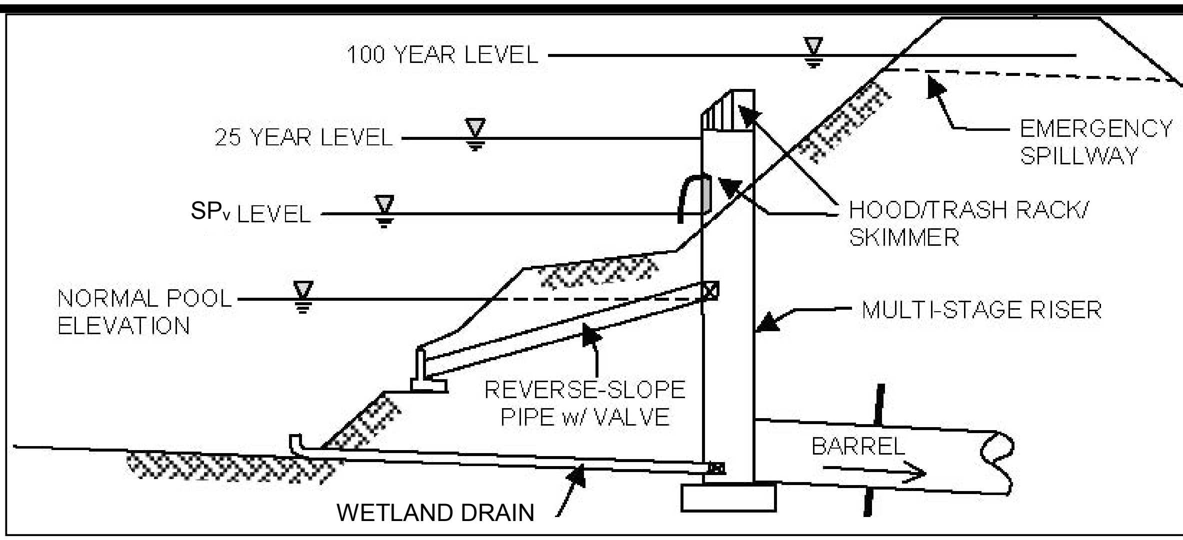


Figure 28.2 Typical Wetland Facility Outlet Structure

- 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 6.0 of the Hydraulic Technical Manual* 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 *Sections 2.0 and 2.2 of the Hydraulics Technical Manual* for additional information and specifications on pond routing and outlet works.

G. Emergency Spillway

An emergency spillway is to be included in the stormwater 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 flood mitigation storm 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 *iSWM Program Guidance – Dams and Reservoirs in Texas*).

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 stormwater 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 the *Landscape Technical Manual.*

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.

28.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Volume (WQ_v), Streambank protection Volume (SP_v), and the flood mitigation storm Flood Discharge, (for ED wetlands the design volume should be increased by 15%).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual.*

Step 2 Determine if the development site and conditions are appropriate for the use of a stormwater wetland

Consider the Application and Site Feasibility Criteria in *Sections 28.4 and 28.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 28.5* (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 28.1.

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 *Section 28.5* (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 flood mitigation stormwater surface elevation, set top of embankment elevation, and analyze safe passage of the flood mitigation storm event (Q_f).

At final design, provide safe passage for the flood mitigation storm event. Attenuation may not be required.

Step 10 Investigate potential pond/wetland hazard classification

The design and construction of stormwater management ponds and wetlands are required to follow the latest version of the State of Texas Administrative Code for dams and reservoirs (see *iSWM Program Guidance – Dams and Reservoirs in Texas*).

Step 11 Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See *Section 28.5* (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 *Section 28.5* (J) (Landscaping) and the *Landscape Technical Manual* for more details.

28.7 Inspection and Maintenance Requirements

Table 28.2 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 stormwater 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 stormwater 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 stormwater 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 stormwater 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 stormwater 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.

28.8 Example Schematics

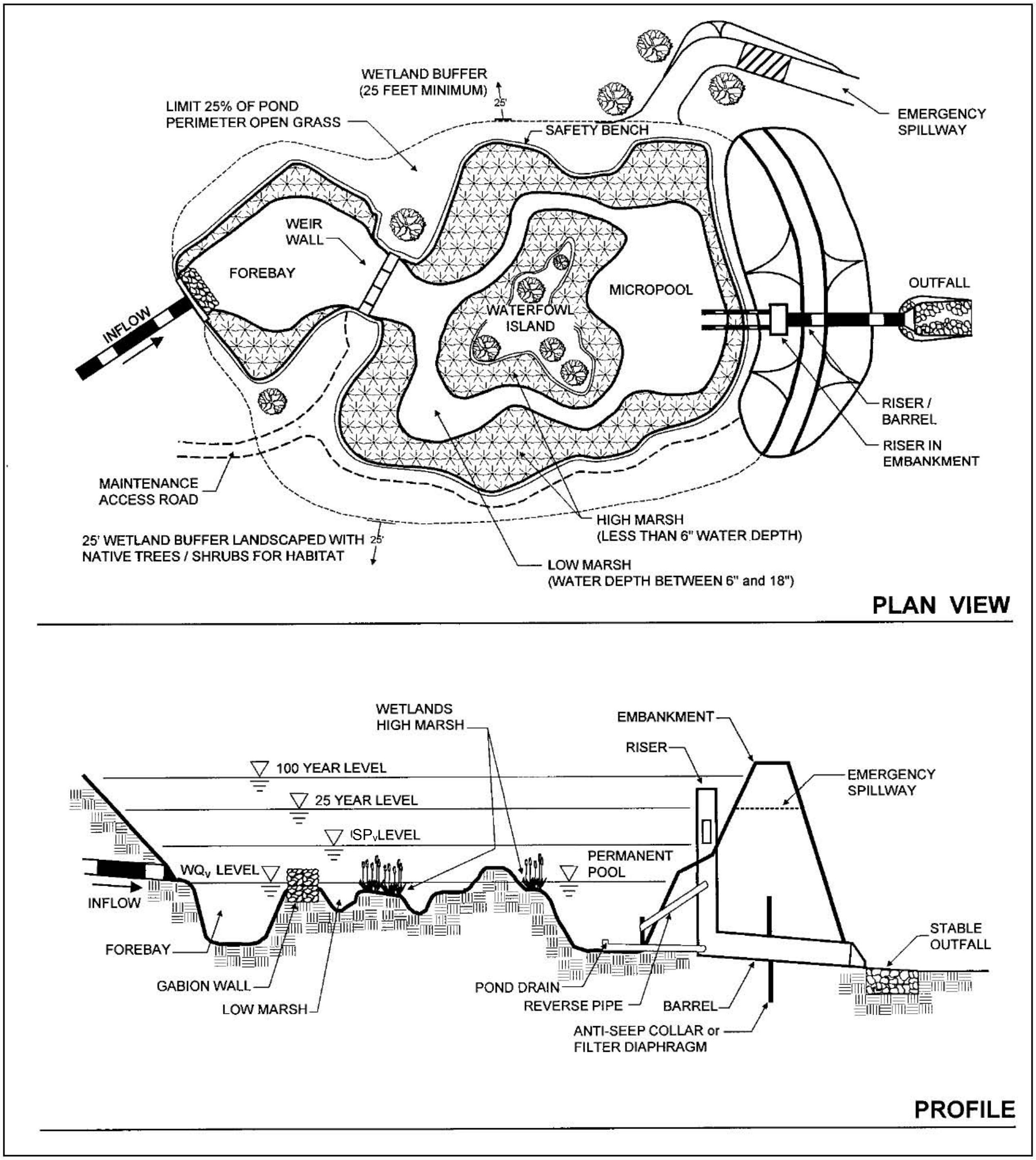


Figure 28.3 Schematic of Shallow Wetland
(Source: Center for Watershed Protection)

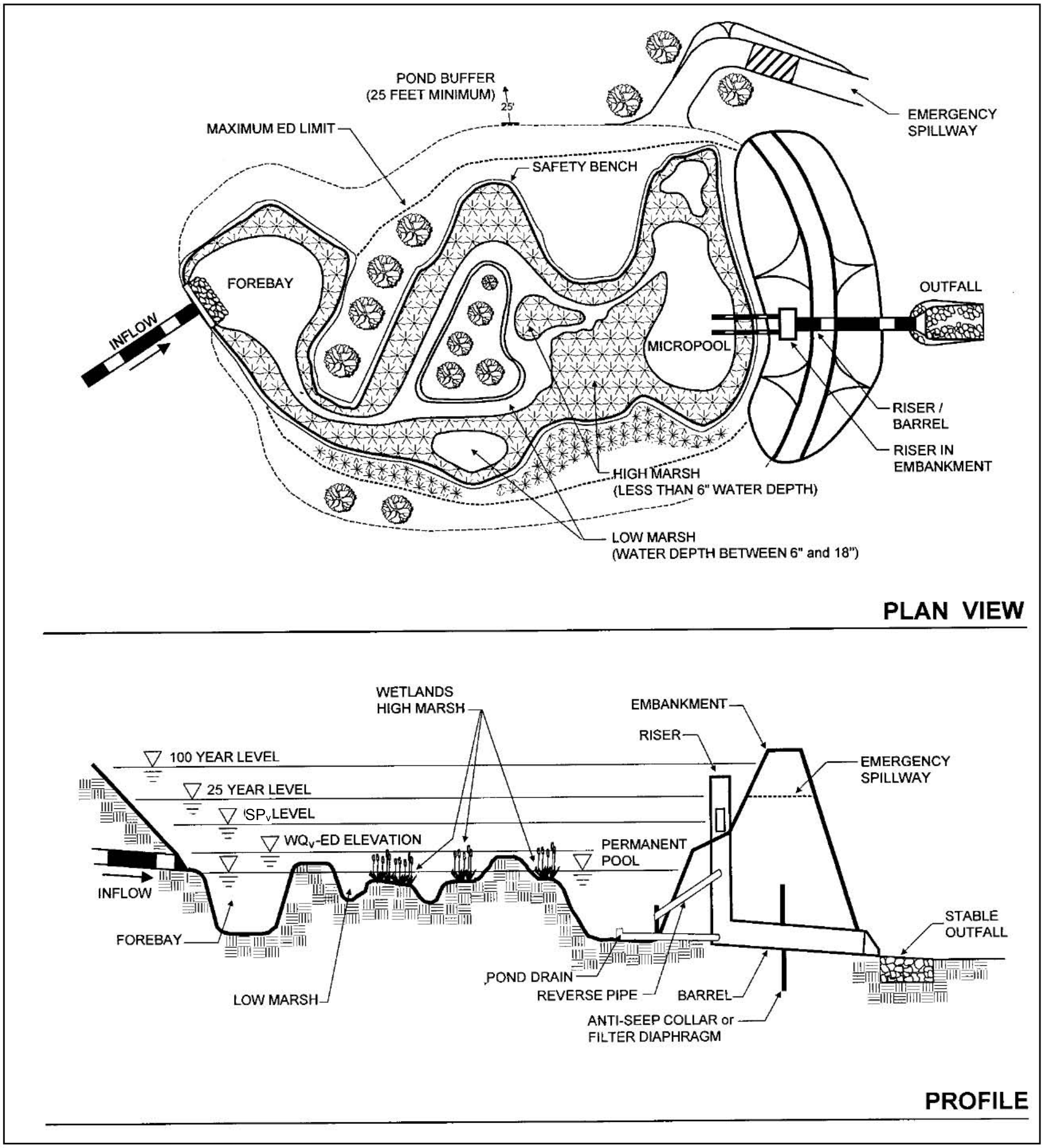


Figure 28.4 Schematic of Extended Detention Shallow Wetland
(Source: Center for Watershed Protection)

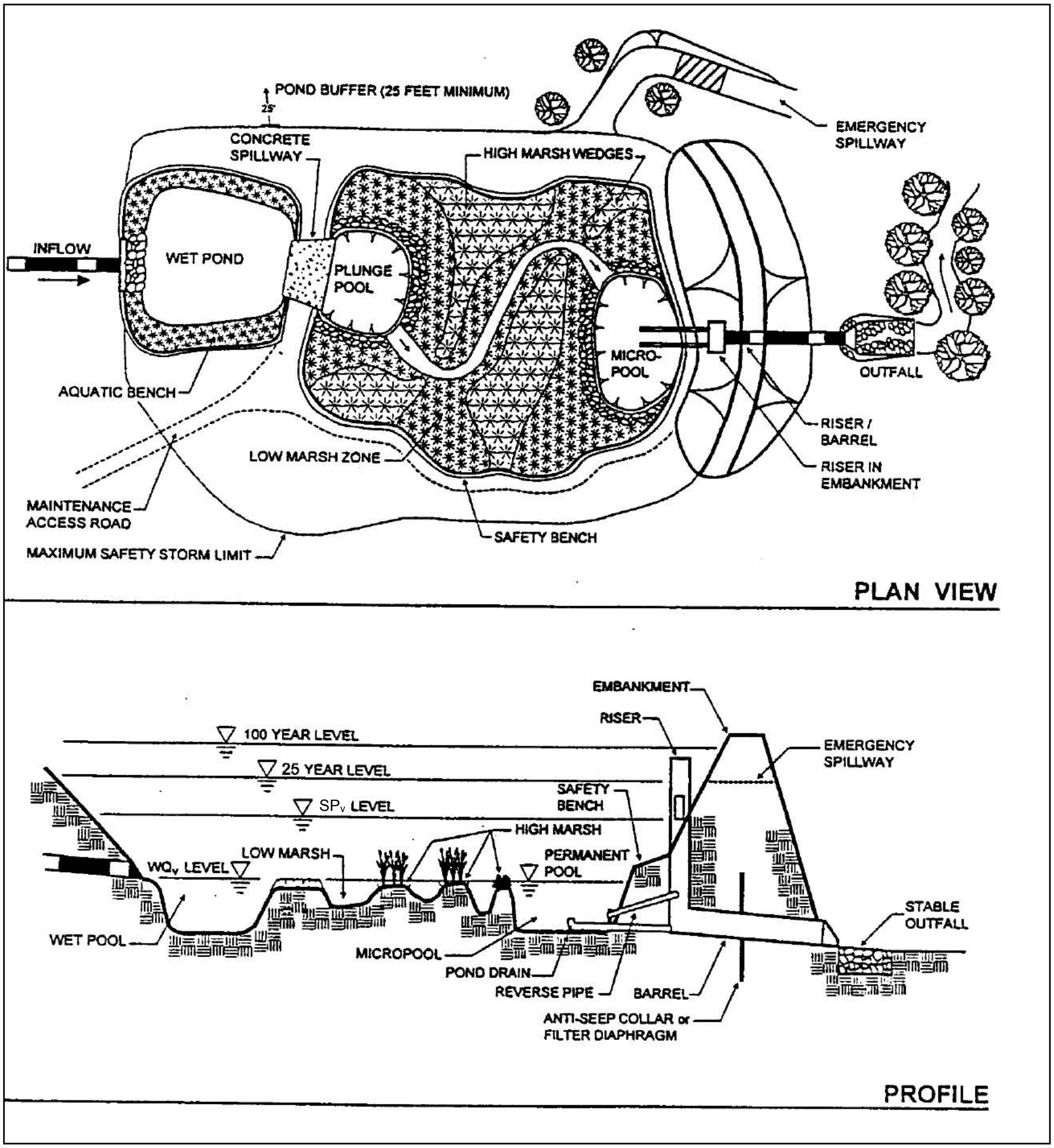


Figure 28.5 Schematic of Pond/Wetland System
(Source: Center for Watershed Protection)

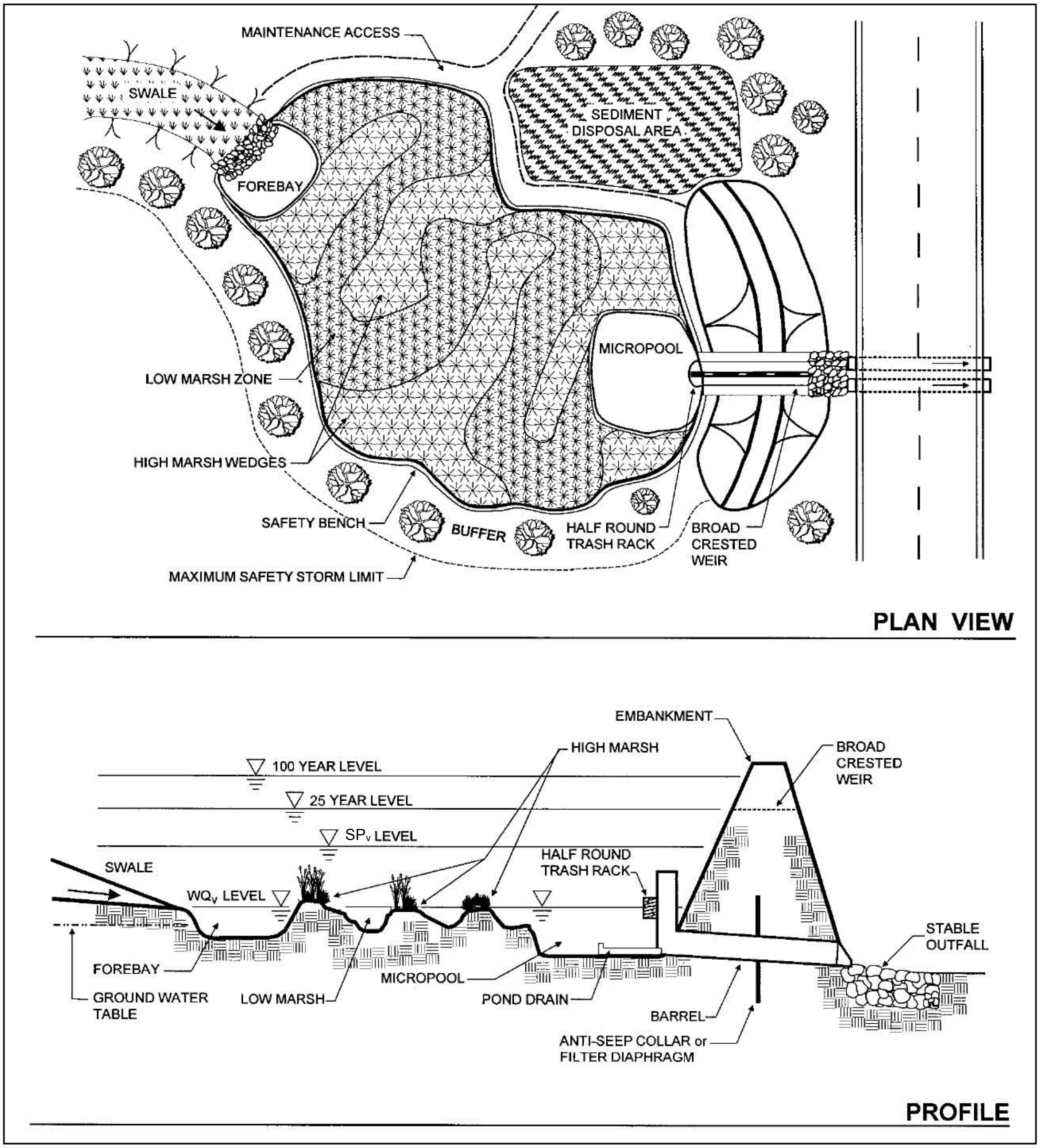
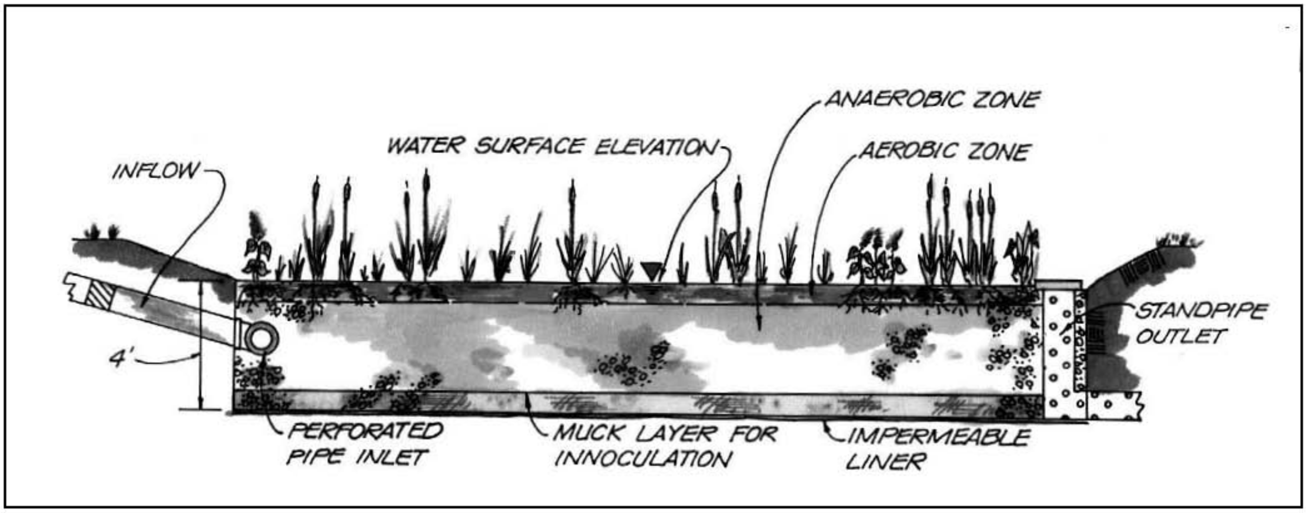


Figure 28.6 Schematic of Pocket Wetland
(Source: Center for Watershed Protection)



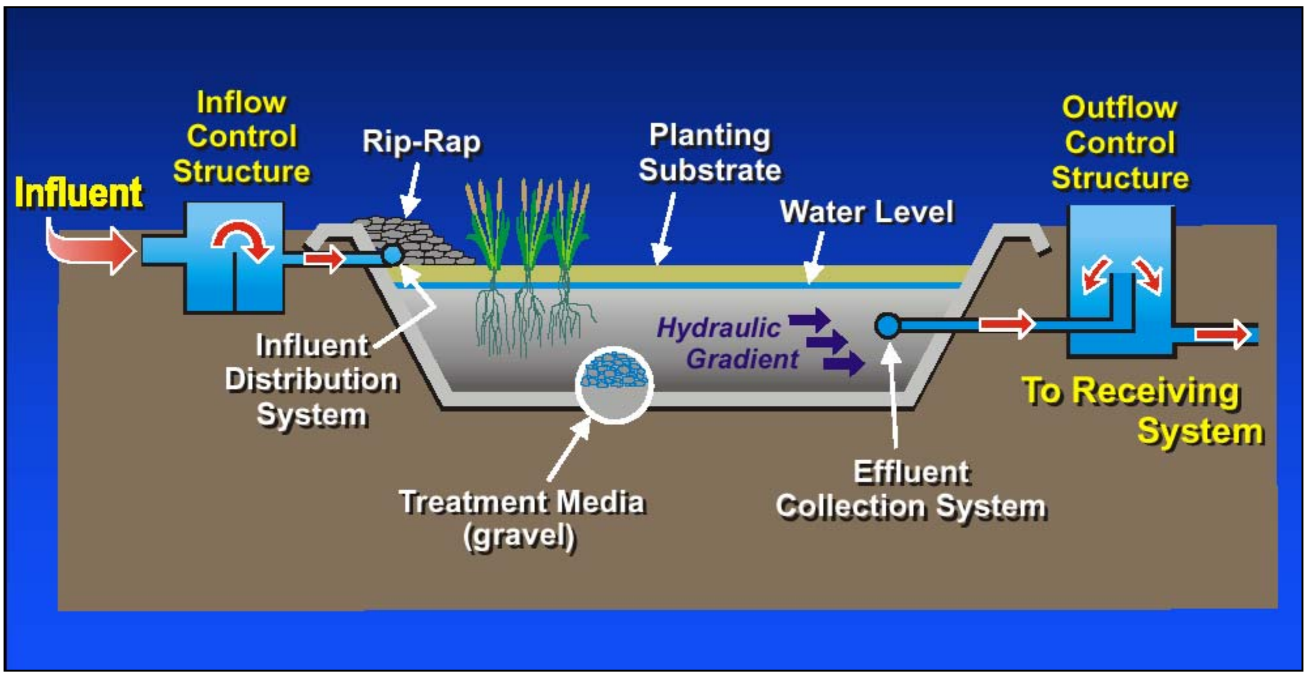


Figure 28.7 Schematics of Submerged Gravel Wetland System
(Sources: Center for Watershed Protection; Roux Associates Inc.)

28.9 Design Forms

Design Procedure Form: Storm Water Wetlands

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements
Compute Runoff Coefficient, R_v — R_v = ____________
Compute WQ_v — WQ_v = ____________ acre-ft

1b. Compute SP_v — SP_v = ____________ acre-ft
Compute average release rate — release rate = ____________ cfs
Compute Q_p (100-year detention volume required) — 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 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_{pool} = ____________ acre-ft
Vol_{marsh} = 0.75 (WQ_v) — Vol_{marsh} = ____________ acre-ft

Shallow ED Wetland: Vol_{pool} = 0.25 (WQ_v) — Vol_{pool} = ____________ acre-ft
Vol_{marsh} = 0.25 (WQ_v) — Vol_{marsh} = ____________ acre-ft
Vol_{ED} = 0.5 (WQ_v) — Vol_{ED} = ____________ acre-ft

Pocket Wetland: Vol_{pool} = 0.25 (WQ_v) — Vol_{pool} = ____________ acre-ft
Vol_{marsh} = 0.75 (WQ_v) — 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) — $Area_{water}$ = ____________ acres, % = ________
Low Marsh Wetland Zone (6-18 inches deep) — $Area_{low}$ = ____________ acres, % = ________
High Marsh Wetland Zone (0-6 inches deep) — $Area_{high}$ = ____________ acres, % = ________
Semi-Wet Wetland Zone (above pool depth) — $Area_{semi}$ = ____________ 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 ft²	Average Area ft²	Depth ft	Volume ft³	Cummulative Volume ft³	Cummulative Volume ac-ft	Volume above Permanent Pool ac-ft

Notes:

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
$Q = CA(2gh)^{0.5}$ — A = ____________ ft^2
diameter = ____________ in
Discharge equation $Q = (h)^{0.5}$ — factor = ____________ $(h)^{0.5}$

Compute release rate for SP_v-ED control and establish SP_v elevation — WSEL = ____________ ft-NGVD
Release rate = — release rate = ____________ cfs
Average head, h = (SP_v elev. - Permanent pool elev.)/2 — h = ____________ ft
Area of orifice from orifice equation — A = ____________ ft^2
$Q = CA(2gh)^{0.5}$ — diameter = ____________ in
Discharge equation $Q = (h)^{0.5}$ — factor = ____________ $(h)^{0.5}$

8. Calculate Q_p release rate and WSEL — ***Set up a stage-storage-discharge relationship***

Elevation	Storage	Low Flow	Riser			Barrier		Emergency	Total
		WQv-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) — Q_p = ____________ cfs

Maximum head = — H = ____________ ft
Use weir equation for slot length ($Q = CLH^{3/2}$) — L = ____________ ft

Check inlet condition
Check outlet condition — ***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:

29.0 Stormwater Control Design Examples

29.1 Introduction

The five stormwater control examples included in this section are presented to provide example computations using the most commonly used design standards. All of the possible options that might be incorporated into a stormwater control structure are not covered in these examples.

For all possible design standards and options that might be incorporated into the design of a stormwater control structure, refer to *Table 1.1 of the Planning Technical Manual*. In order to apply design standards to a given stormwater control structure, additions or modifications to *Section 1.0 and 2.0 of the Planning Technical Manual* must also be considered. These are found in the Criteria Manual.

The Criteria Manual may determine the extent of downstream assessments; the return periods to be used for the "Streambank Protection" and "Conveyance" storms; and the acceptable design focus area 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 stormwater control structure.

29.2 Stormwater Pond Design Example

The following design example is for a wet extended detention (ED) stormwater 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 29.1.

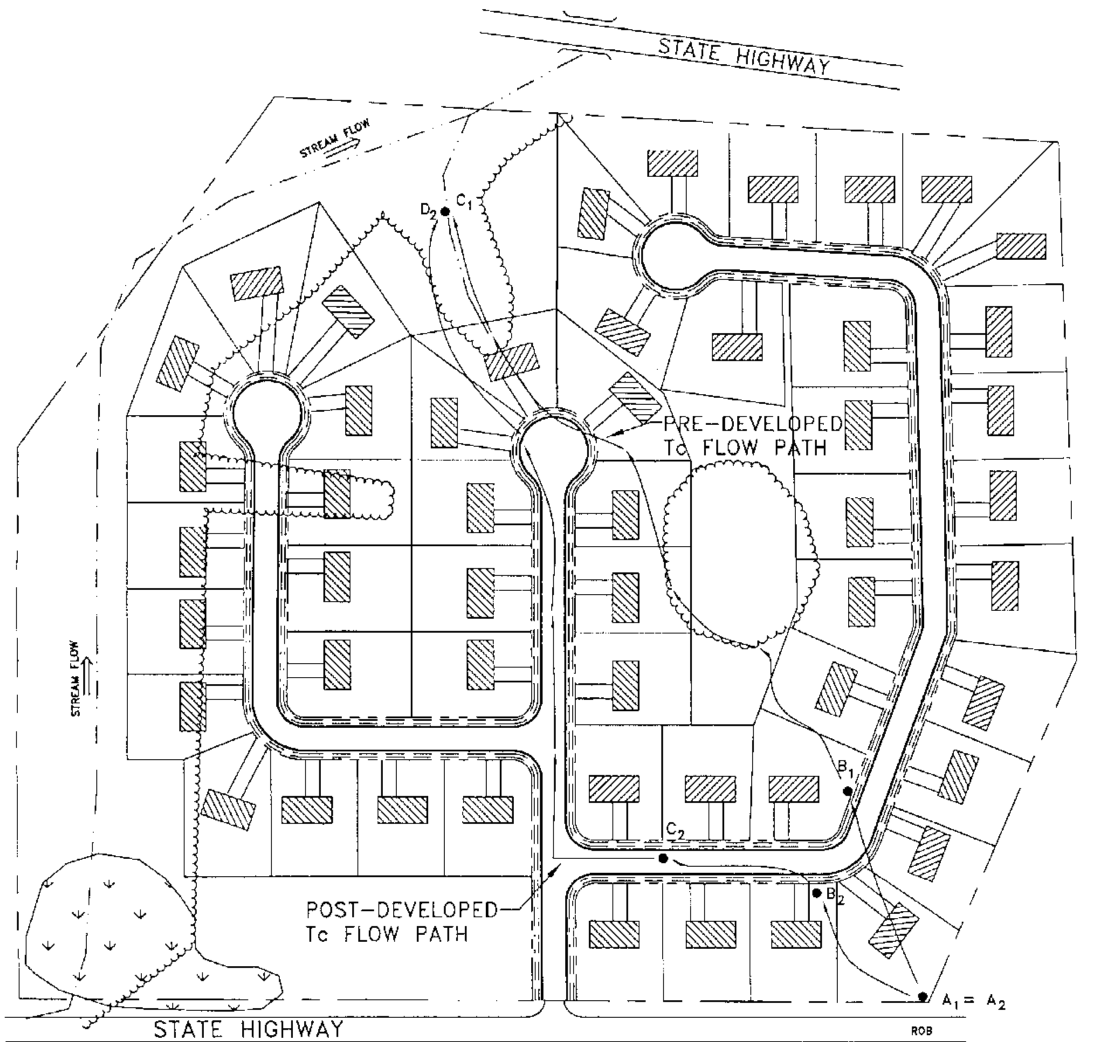


Figure 29.1 Rolling Meadows Site Plan

Base Data	Hydrologic Data		
		Pre	Post
Site Area = Total Drainage Area (A) = 38.0 ac	CN	76	85
Measured Impervious Area=13.8 ac; or I=13.8/38=36.3%	t_c	0.33 hr	0.19 hr
Soils Types: 20% "C", 80% "B"			
Zoning: Residential (½ acre lots) Denton County			

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1: Compute runoff control volumes from the integrated Design Focus Areas

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 29.2 and 29.3. 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 1.1 of the Hydrology Technical Manual.*

- 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 29.1 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 5.3 of the Hydrology Technical Manual*, i_{100} =6.99 in/hr
- From *Equation 1.26 of the Hydrology Technical Manual*, $Q_a= c_a\ i\ A= (0.45)\ (6.99)\ (38)=119.5$ cfs

Determine the Critical Duration of the Storm T_d

- From *Table 1.18 of the Hydrology Technical Manual*, a=325.18; b=24.822
- Post-developed Rational Coefficient, c=0.61
- From *Equation 1.27 of the Hydrology Technical Manual*,

  $Td=\sqrt{(2cAab/Q_a)} - b$

  $Td=\sqrt{(2(0.61)(38)\ (325.18)\ (24.822)/(119.5)\ )} - (24.822) = 31.1$ min
- From *Table 5.3 of the Hydrology Technical Manual*, 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 5.3 of the Hydrology Technical Manual*, i_{100} =6.99 in/hr
- From *Equation 1.26 of the Hydrology Technical Manual*,

  $Q_a= c_a\ i\ A= (0.45)\ (6.99)\ (38)=119.5$ cfs

<u>Compute Storage Volume</u>

- Post-developed Time of concentration, t_c = 0.19 hr (60 min/hr) = 11.4 min
- From *Equation 1.28a of the Hydrology Technical Manual*,

  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 1.28b of the Hydrology Technical Manual*,

  V_{max} = V_{pre} * P_{180}/P_{Td}

  P_{180} = 1.79 in/hr (3 hr) in *Table 5.3 of the Hydrology Technical Manual.*

  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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.2 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.3 Rolling Meadows Post-Development Conditions

Step 2: Determine if the development site and conditions are appropriate for the use of a stormwater 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 1.1 and 1.2 of the Hydraulics Technical Manual* 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:

$0.5 \times$ (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: $0.5 \times$ (1.44 ac-ft) = **0.72 ac-ft**

Note: This design focus area 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 29.4 for pond location on site, Figure 29.5 grading and Figure 29.6 for Elevation-Storage Data.

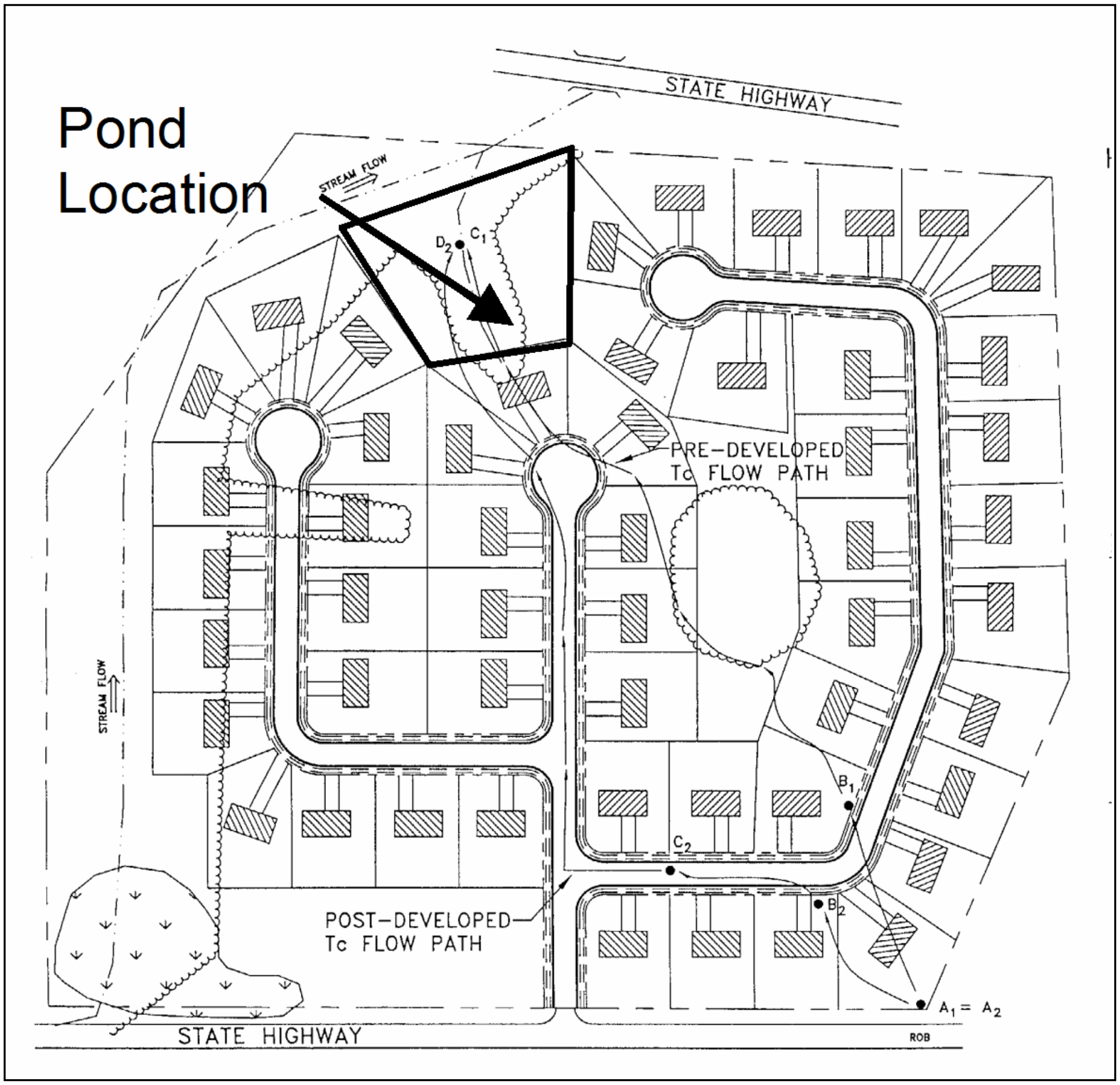


Figure 29.4 Pond Location on Site

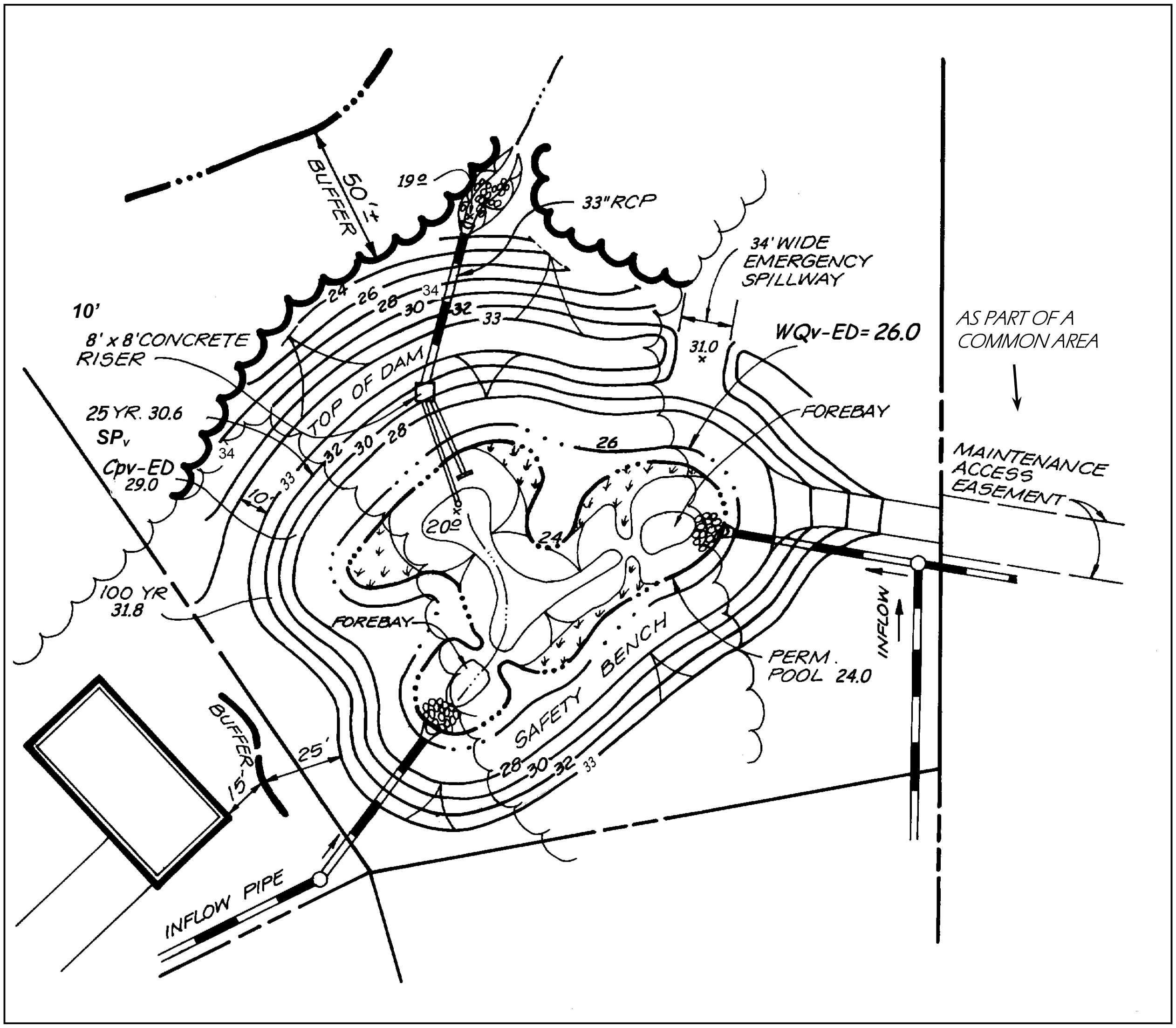


Figure 29.5 Plan View of Pond Grading (Not to Scale)

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

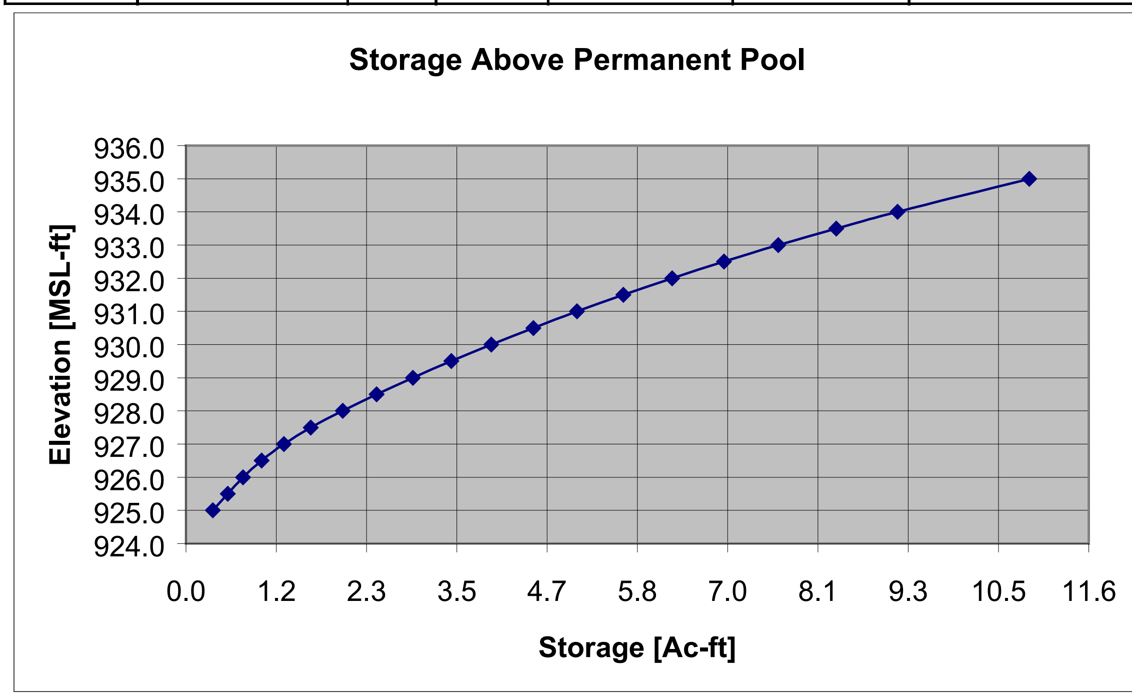


Figure 29.6 Storage-Elevation Table and Curve

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 cfs / [(0.6)((2)32.2 ft/s^2)(1.0 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

- $Q_{WQv-ED} = CA(2gh)^{0.5}$ = (0.6) (0.075 ft^2) [((2)(32.2 ft/s^2))$^{0.5}$] ($h^{0.5}$),
- Q_{WQv-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 29.1).
- 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 29.1,
- From previous estimate Q_f = 4.88 ac-ft.
- From elevation-storage table (Figure 29.6), 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

<u>Size barrel to release approximately 266 cfs at elevation 930.9</u>

- Check inlet condition: (use *Section 3.3 of the Hydraulic Technical Manual*)
  - H_w = 930.9-919.5 = 11.4 ft
  - Try 60" diameter RCP, Using *Figure 1.19a of the Hydraulics Technical Manual* 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 29.7) 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: SPv-ED 4.7" eq dia		Riser: High Stage Slot: Orifice		Riser: High Stage Slot: Weir		Barrel: Inlet		Barrel: Pipe		Emergency Spillway		Total Discharge
		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 29.7 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 29.1, 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 29.2 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-100})	34' wide earth spillway	6.53	932.2	207	volume above perm. pool

See Figure 29.8 for profile through principal spillway of the facility.

See Figure 29.9 for a schematic of the riser.

Step 12: Prepare Vegetation and Landscaping Plan

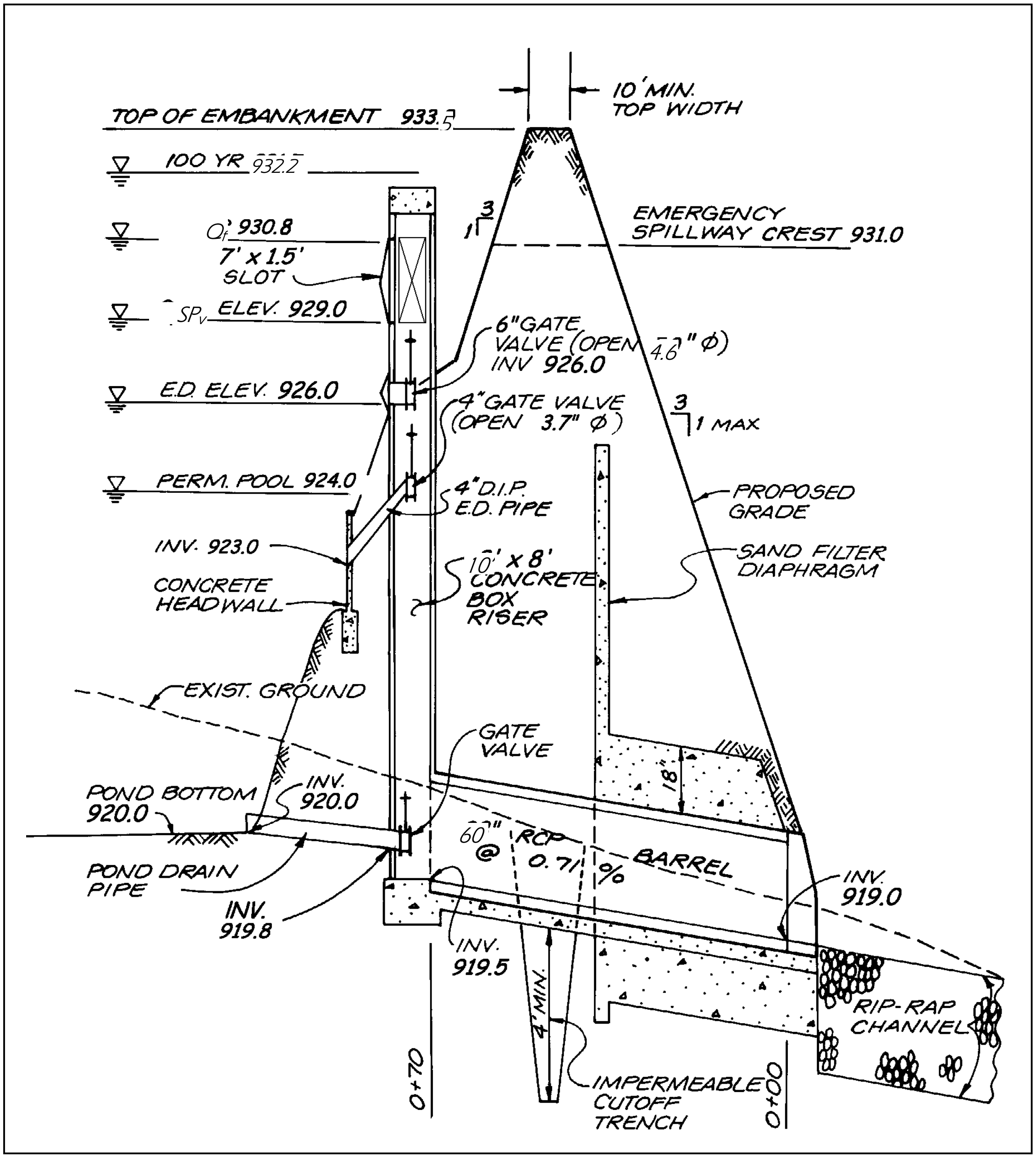


Figure 29.8 Profile of Principal Spillway

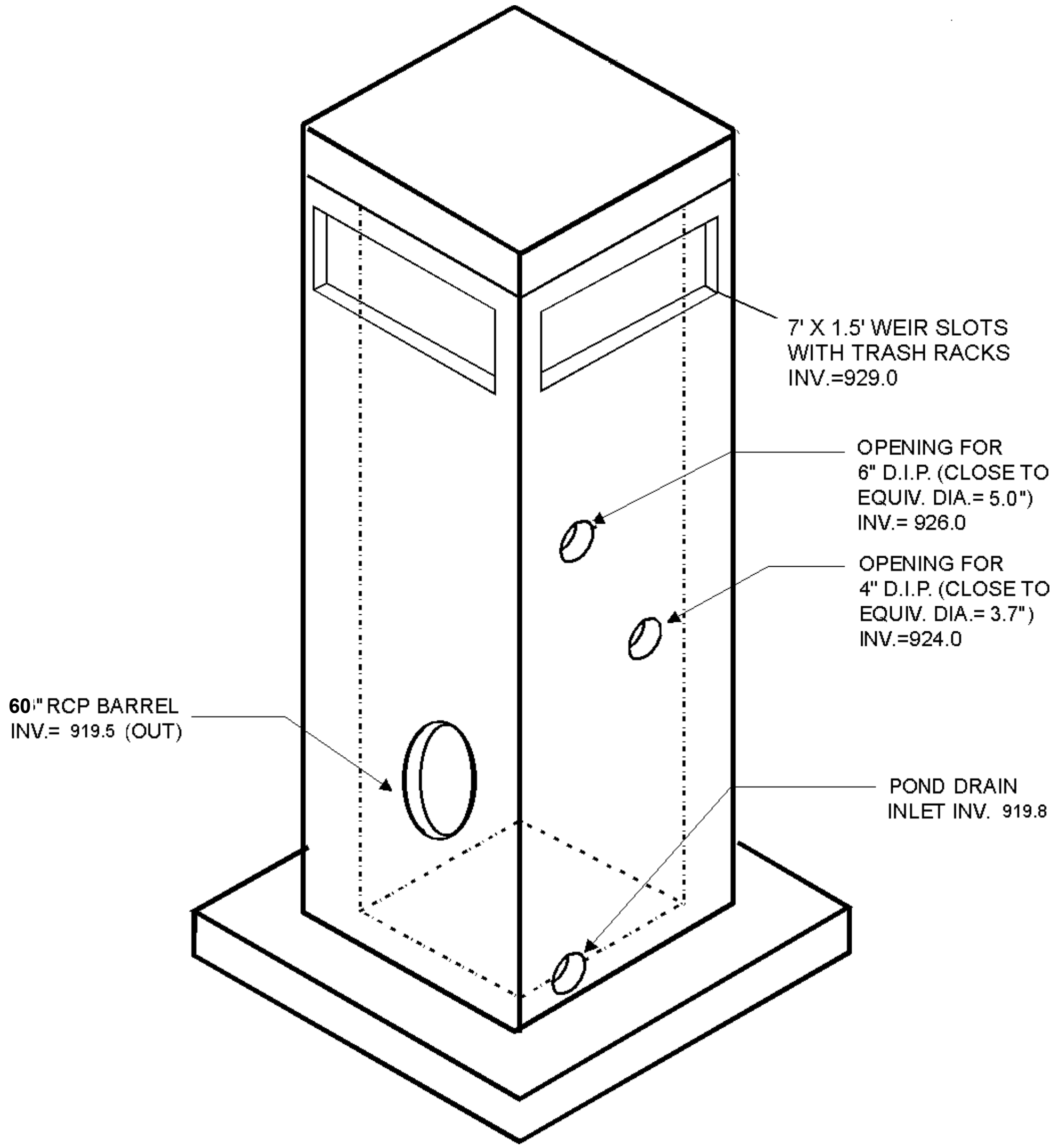


Figure 29.9 Schematic of Riser Detail

29.3 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 29.10.

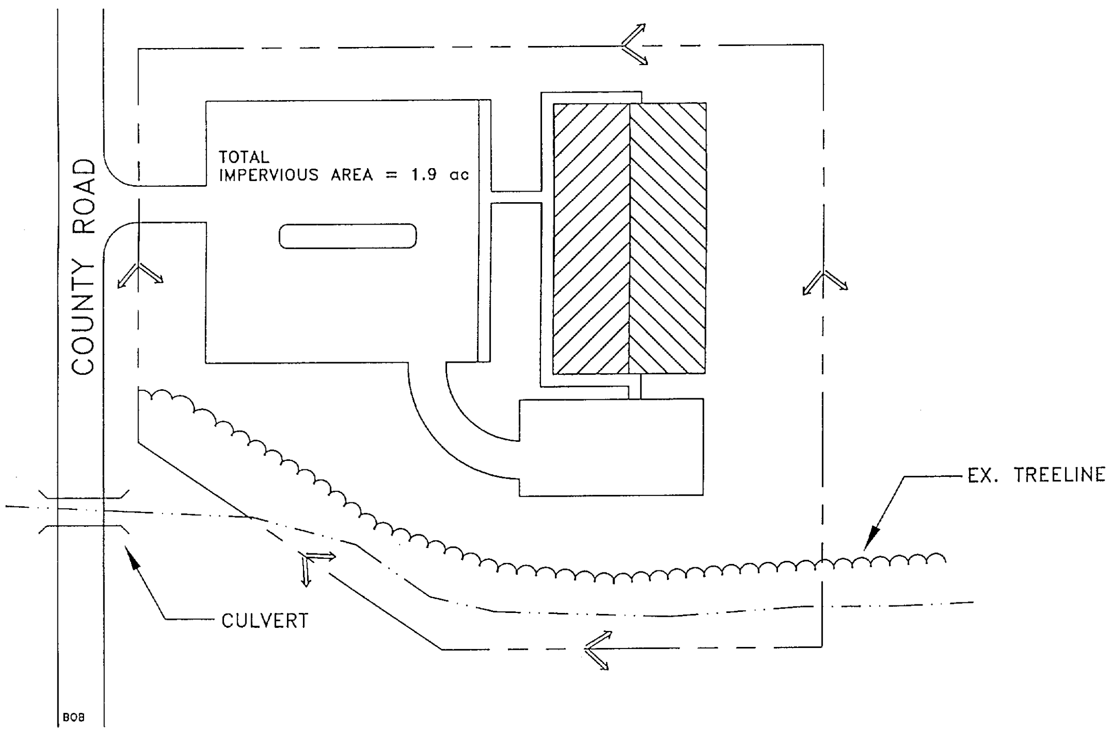


Figure 29.10 Wellington Recreation Center Site Plan

Base Data	Hydrologic Data
Site Area = Total Drainage Area (A) = 3.0 ac Measured Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3% Soils Type "D" Collin County	Pre Post CN 77 91 t_c 0.41 hr 0.20 hr

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1: Compute runoff control volumes from the integrated Design Focus Areas

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.5")(R_v)(A) / 12$

  $= (1.5")(0.62)(3.0\text{ac})(43{,}560\text{ft}^2/\text{ac})(1\text{ft}/12\text{in})$

  $= \underline{8{,}102}\ \text{ft}^3$

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 1.1 of the Hydrology Technical Manual* 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 29.11 and 29.12)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\text{ year}}$
		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: [$I_a = (200/CN - 2)$]

  $I_a/P = (0.198)/ 2.64 \text{ inches} = 0.075$

  $T_c = 0.20$ 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 29.3 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.11 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.12 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 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 29.13. 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 29.14 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.

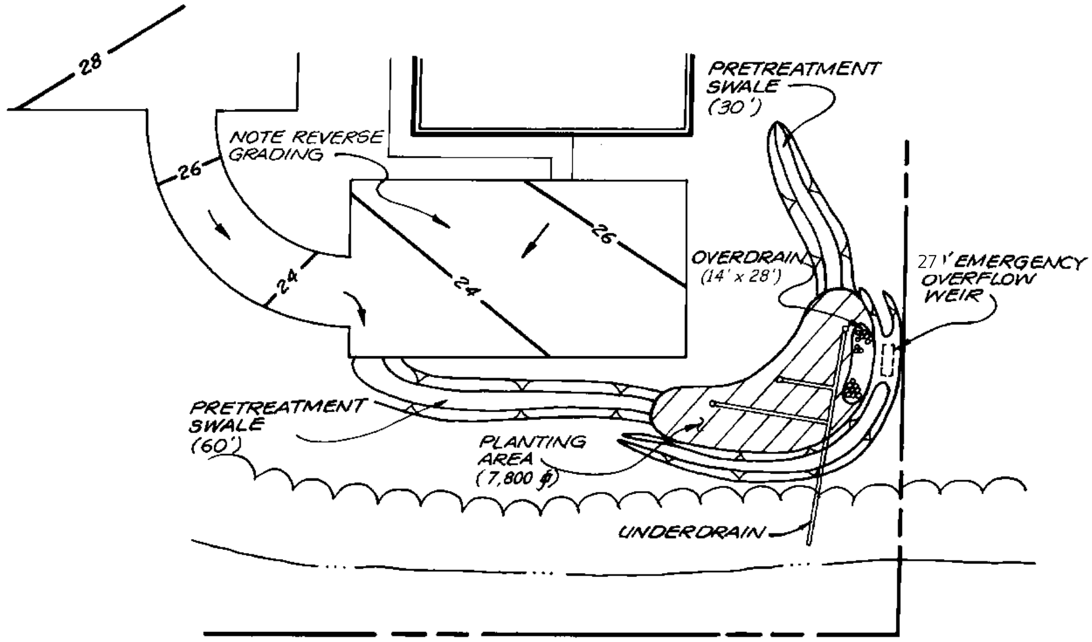


Figure 29.13 Plan View of Bioretention Facility

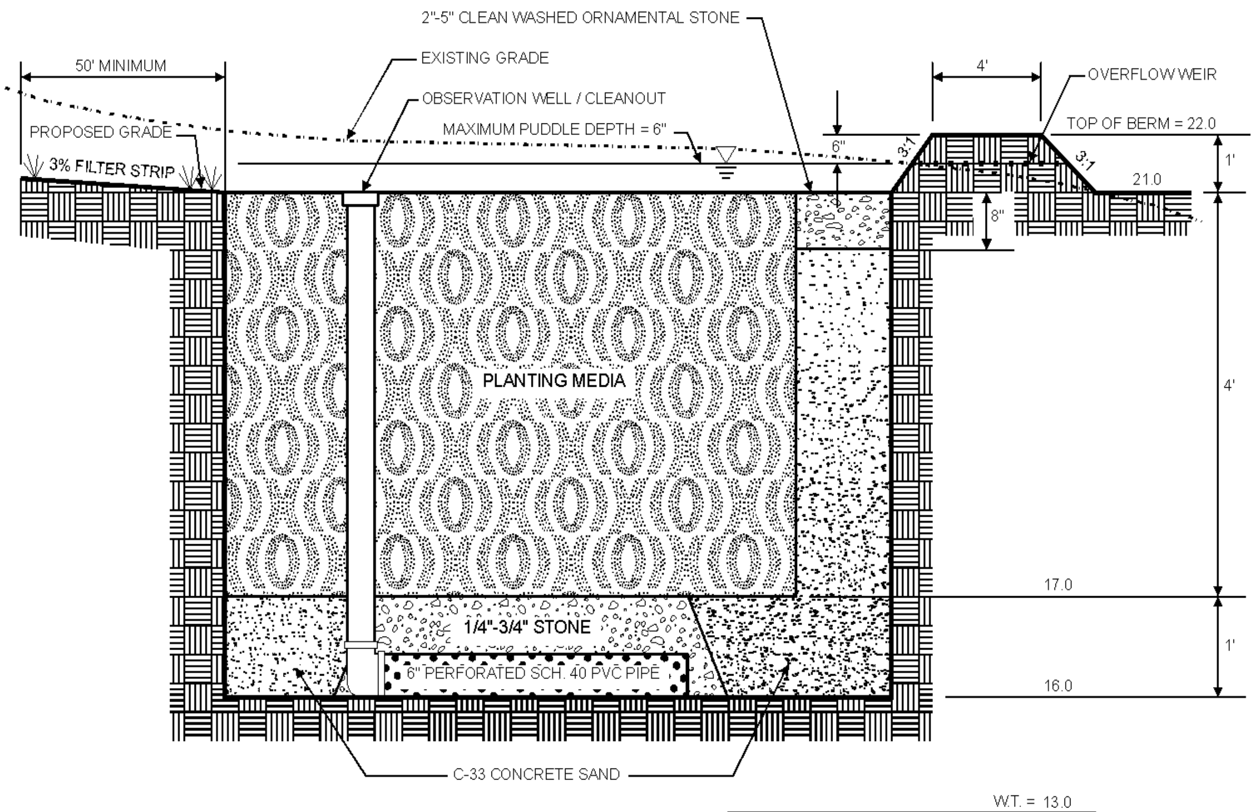


Figure 29.14 Typical Section of Bioretention Facility

Step 9: Design pretreatment

Pretreat with a grass channel, based on guidance provided in Table 29.4, 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 29.4 is 30' for a one acre drainage area.

Table 29.4 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 29.5 and 29.6.

(772 sq ft)/3' per foot of underdrain = 257', say 260' of perforated underdrain

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 29.5 and 29.6.

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 $(25.0 \text{ cfs}) / [(2.65)(.5)^{1.5}] = 26.7'$ (say 27')

Outlet protection in the form of riprap or a plunge pool/stilling basin should be provided to ensure non-erosive velocities. See Figures 29.5 and 29.6.

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 the *Landscape Technical Manual*.

29.4 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 29.15.

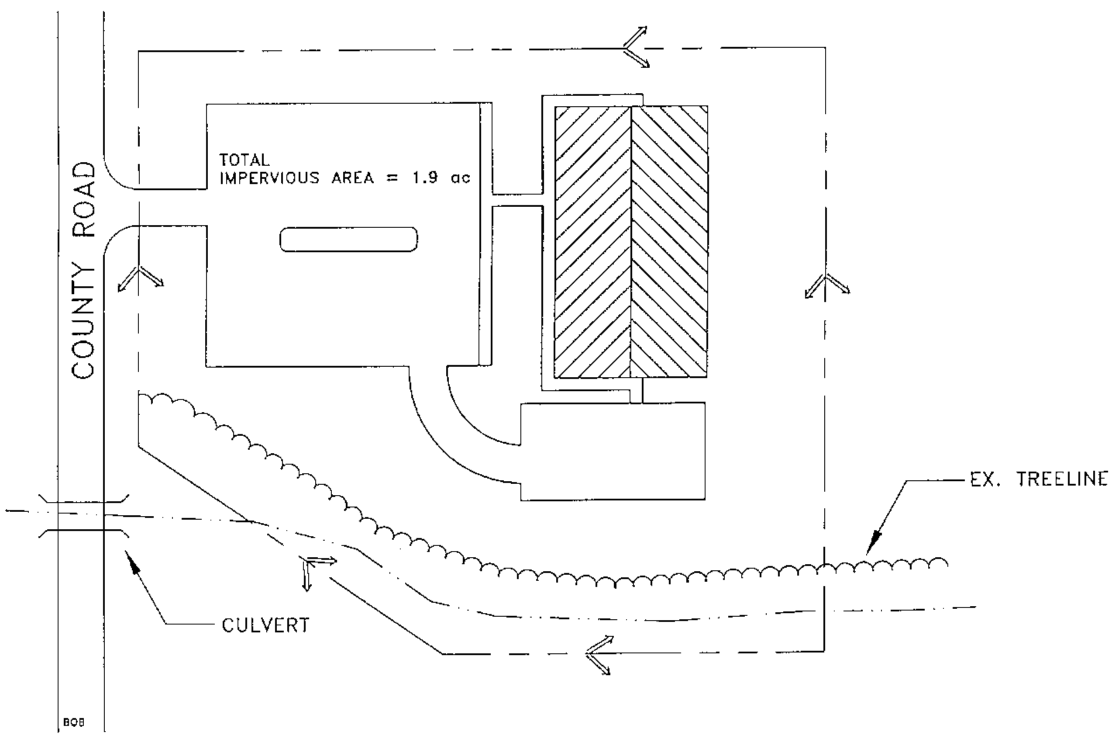


Figure 29.15 Falcon Creek Community Center Site Plan

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 Stormwater Storage Volumes and Peak Discharges

Step 1: Compute runoff control volumes from the *integrated* Design Focus Areas

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 29.16 and 29.17. 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	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 29.5 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.16 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 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**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 29.17 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 29.18)

  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 *Equation 1.8 of the Hydrology Technical Manual*:

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

  where P = rainfall, in inches (use 1.5" for the Water Quality Storm)
  and Q = runoff volume, in inches (equal to WQ_V ÷ 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 1.4 of the Water Quality Technical Manual* (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

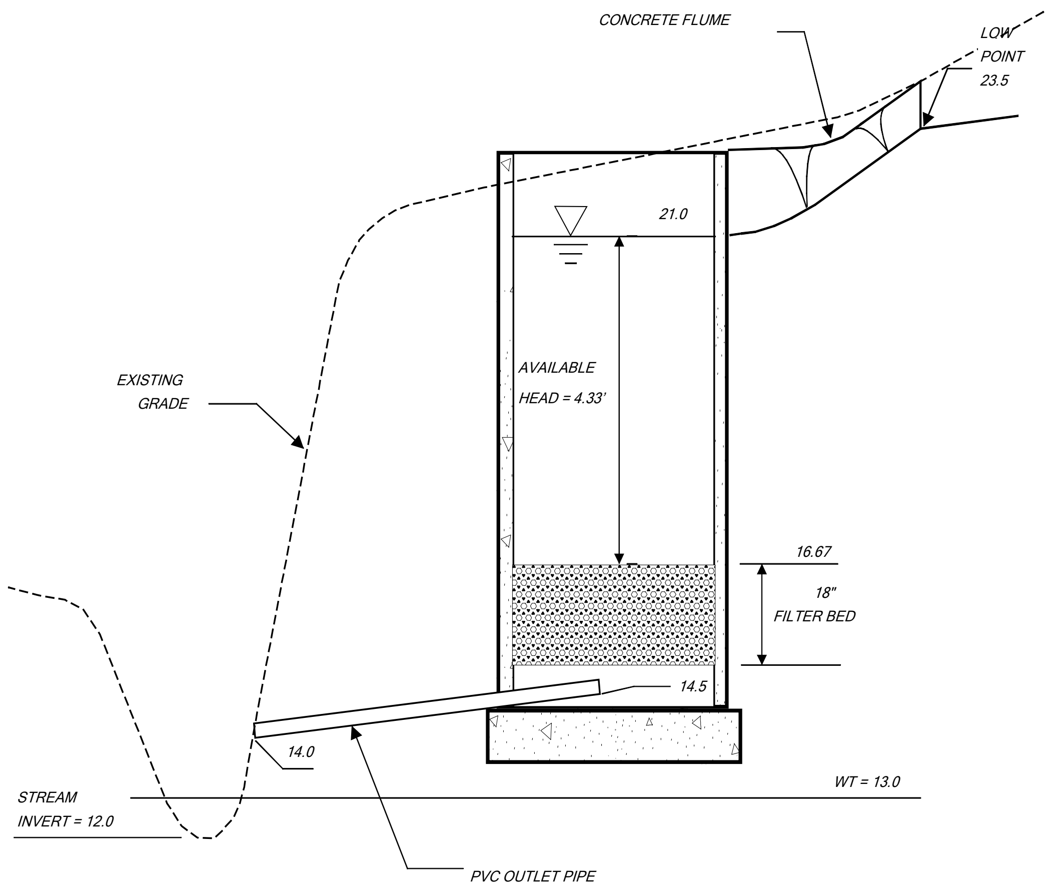


Figure 29.18 Available Head Diagram

- Using the water quality volume (WQ_v), compute the water quality peak discharge (Q_{wq})

  $Q_{wq} = q_u$*A*WQ_V **(29.2)**

  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 ÷ area = (8,102 ft^3 ÷ 3 ac ÷ 43,560 ft^2/ac × 12 in/ft) = 0.74"
CN = $1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}]$
= $1000/[10+5*1.5+10*0.74-10(0.74^2+1.25*0.74*1.5)^{½}]$
= 95.01
<u>Use CN = 95</u>

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 1.0 of the Hydrology*

Technical Manual, 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 29.19):

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 29.20):

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)

100

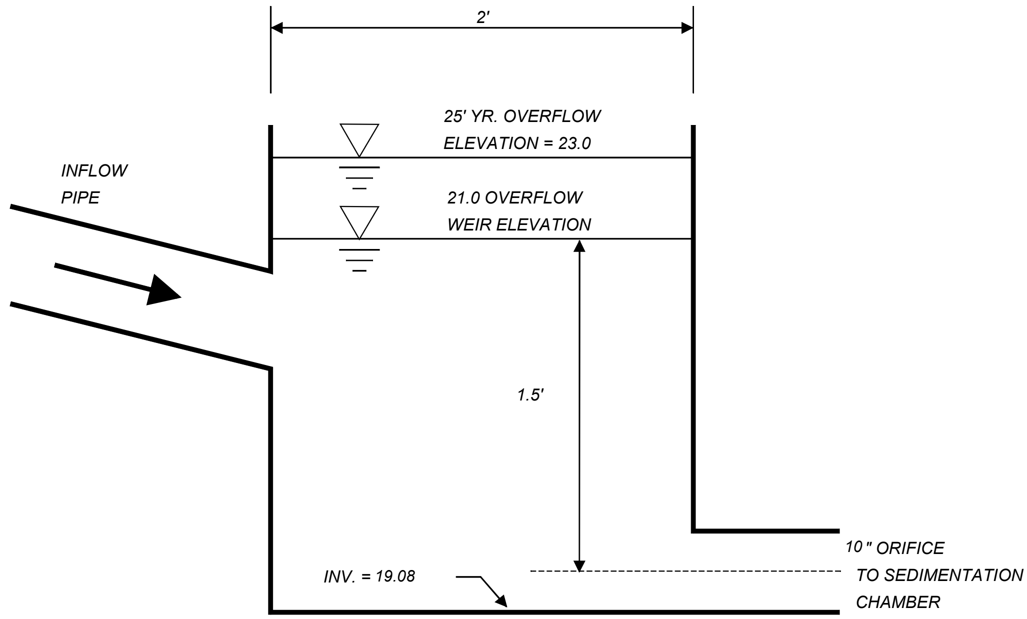


Figure 29.19 Flow Diversion Structure

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 29.21). 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 29.9 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, <u>Use L = 4.75 ft.</u>

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, <u>Use L = 4.75 ft.</u>

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

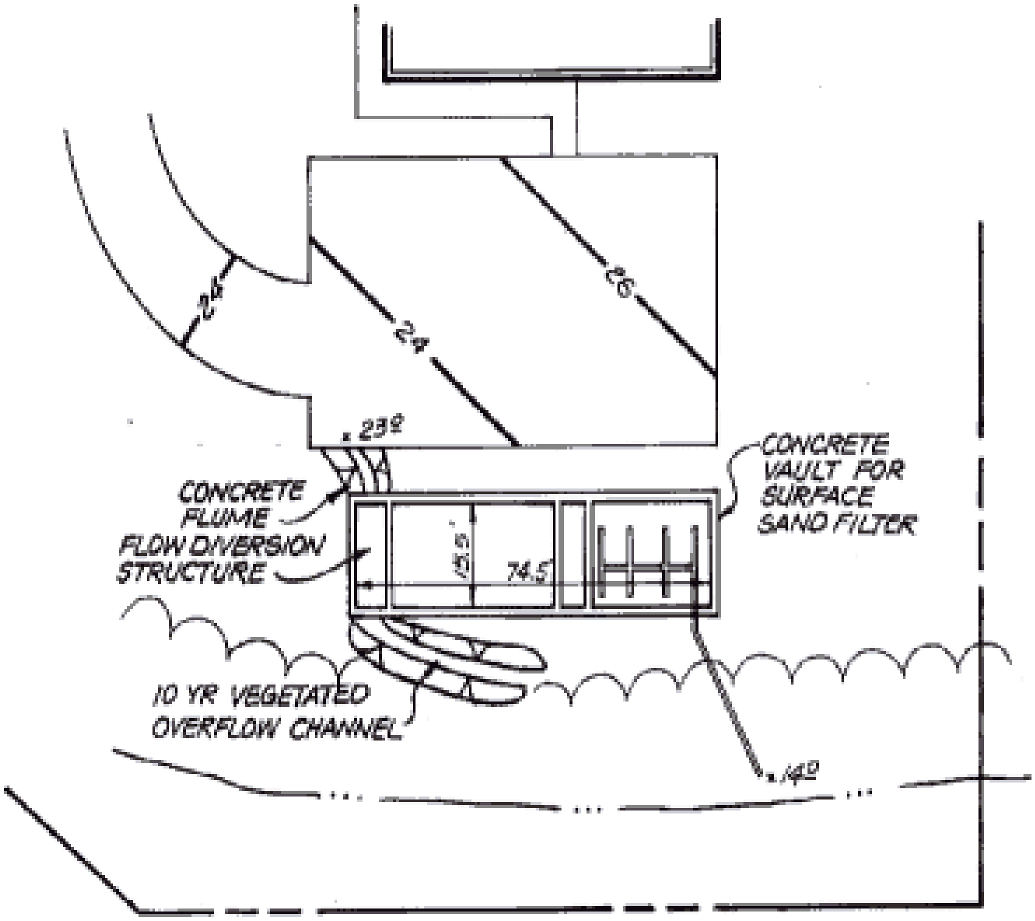


Figure 29.20 Surface Sand Filter Site Plan

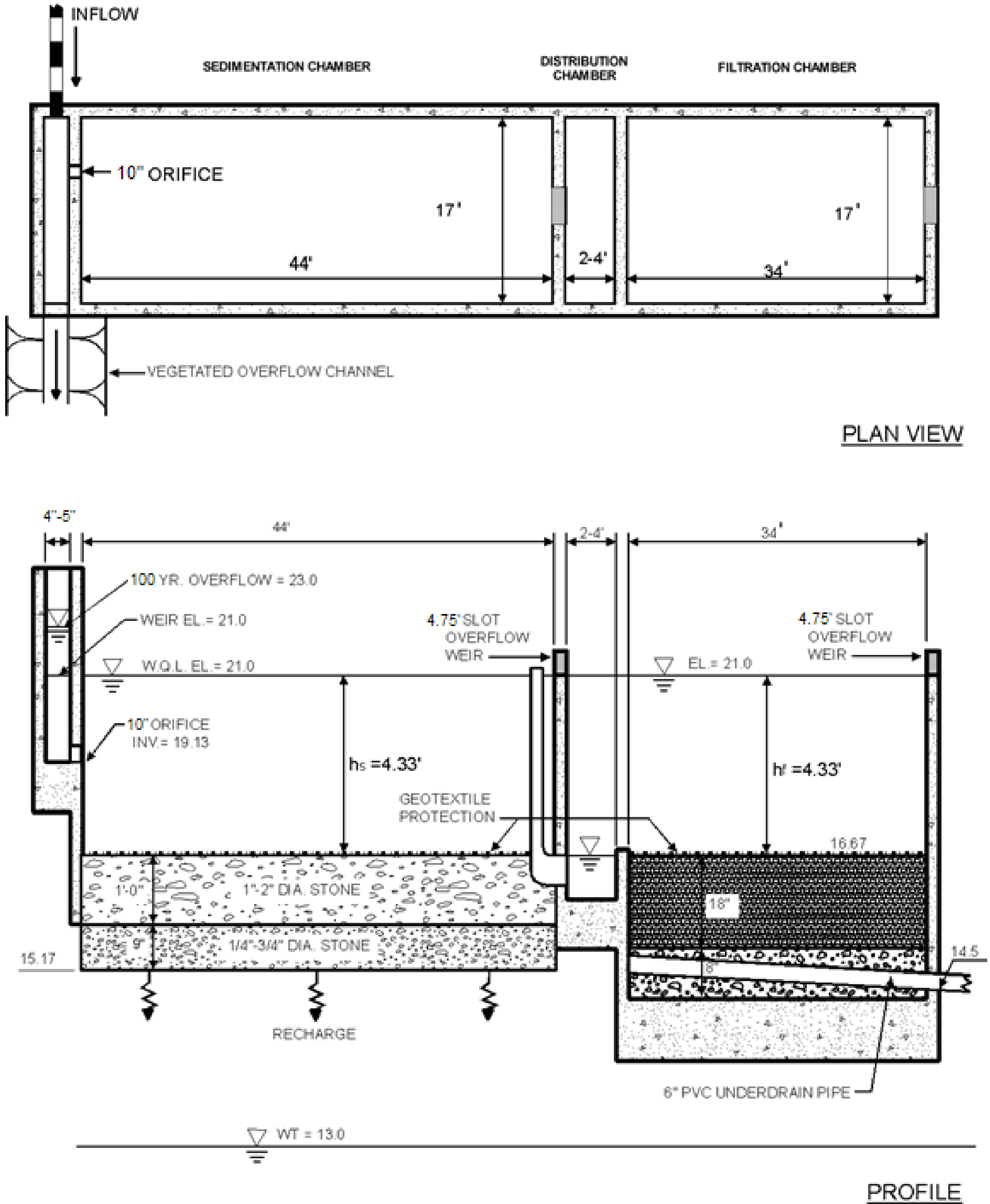


Figure 29.21 Plan and Profile of Surface Sand Filter

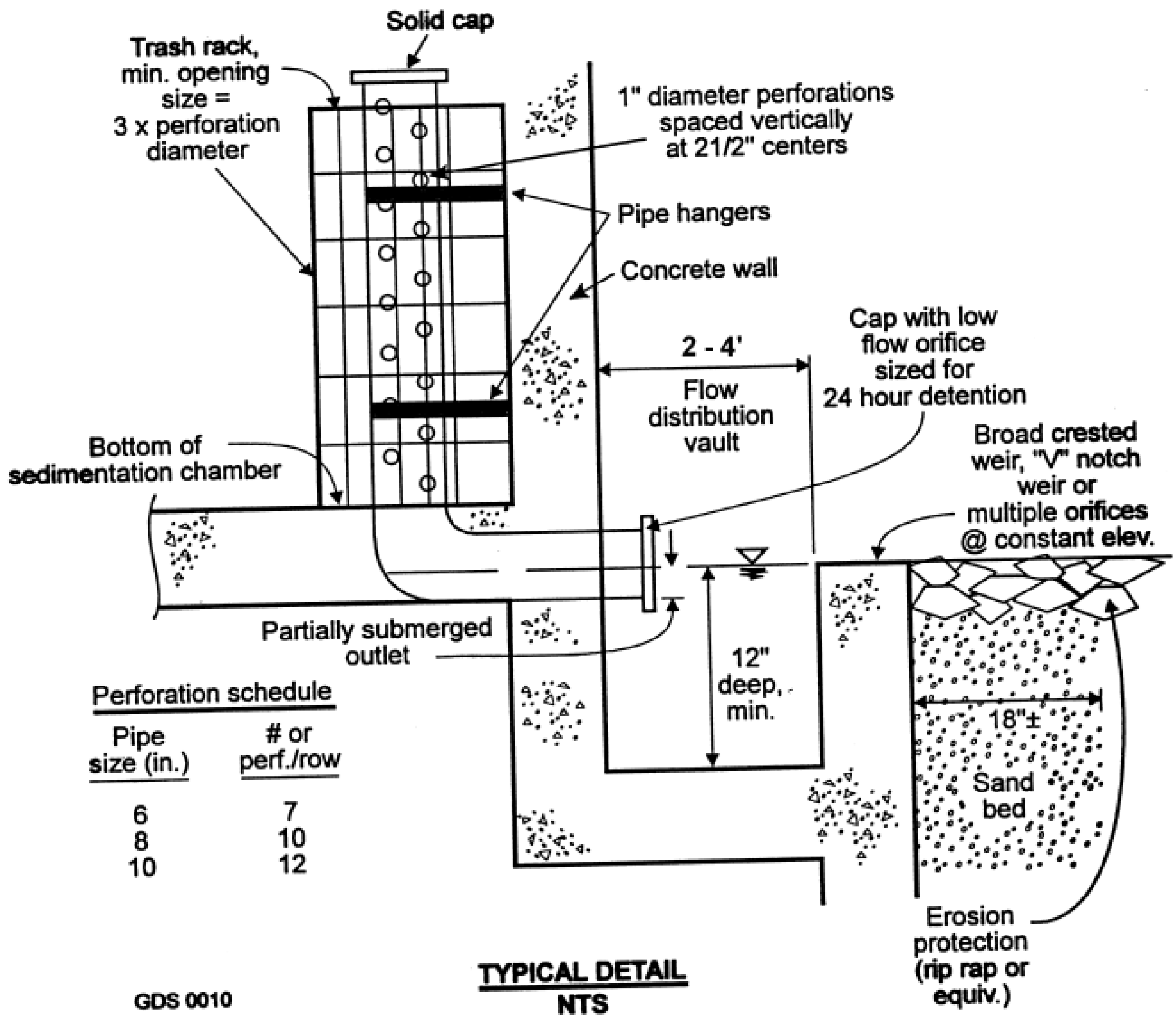


Figure 29.22 Perforated Stand Pipe Detail

29.5 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 29.23.

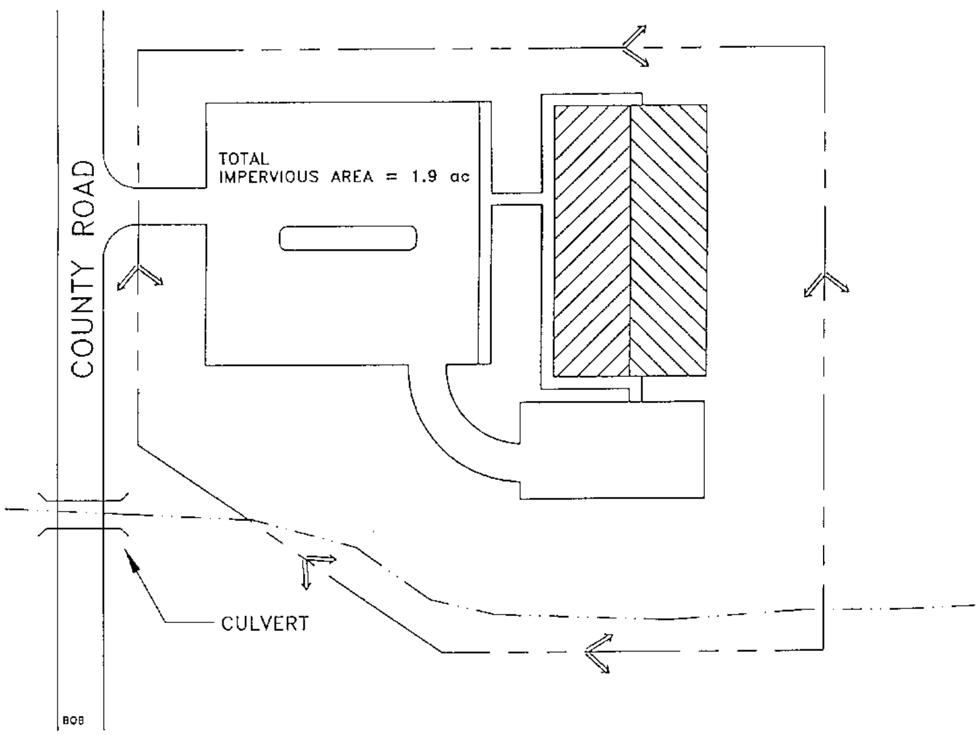


Figure 29.23 Cottonwood Creek Community Center Site Plan

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 Focus Areas

Compute Water Quality Volume (WQ_v):

- <u>Compute Runoff Coefficient, R_v</u>

  R_v = 0.05 + (63.3) (0.009) = 0.62

- <u>Compute WQ_v</u>

  WQ_v = (1.5") (R_v) (A) / 12
  = (1.5") (0.62) (3.0 ac) (43,560 ft^2/ac) (1ft/12in)
  = <u>8,102</u> ft^3 = <u>0.186</u> 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 29.24 and 29.25. 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

- <u>Utilize modified TR-55 approach to compute streambank protection storage volume</u>

  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

- <u>Define the average extended detention Release Rate</u>

  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 29.6 Summary of General Design Info 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 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**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 29.24 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.25 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 29.7 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench.

Table 29.7 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 29.8, 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 29.8 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 ÷ 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 ÷ area = (8,102 ft^3 ÷ 3 ac ÷ 43,560 ft^2/ac × 12 in/ft) = 0.74"

$$\begin{aligned} CN &= 1000/[10 + 5P + 10Q - 10\,(Q^2 + 1.25*Q*P)^{1/2}] \\ &= 1000/[10 + 5*1.5 + 10*0.74 - 10(0.74^2 + 1.25*0.74*1.5)^{1/2}] \\ &= 91.1 \end{aligned}$$

<u>Use CN = 91</u>

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") = <u>2.86 cfs</u>.

Step 5: Size the infiltration trench

The area of the trench can be determined by the following equation (Equation 20.1):

$$A = \frac{WQ_v}{(nd + kT/12)}$$

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:
$A = 8{,}102\ ft^3 / (0.32 \times 5 + 1 \times 2/12)ft$
$A = 4{,}586\ ft^2$

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 29.4 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 29.3)
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.

$Q=CA(2gh)^{1/2}$; $0.95\ cfs = 0.6A(2 \times 32.2\ ft/s^2 \times 1.5')^{1/2}$
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.

$Q = CLH^{1.5}$; $L= Q/(CH^{1.5})$
$L = 8.75\ cfs/ (3.1)(1)^{1.5} = 2.8'$; use 2.8' (see Figure 29.27)

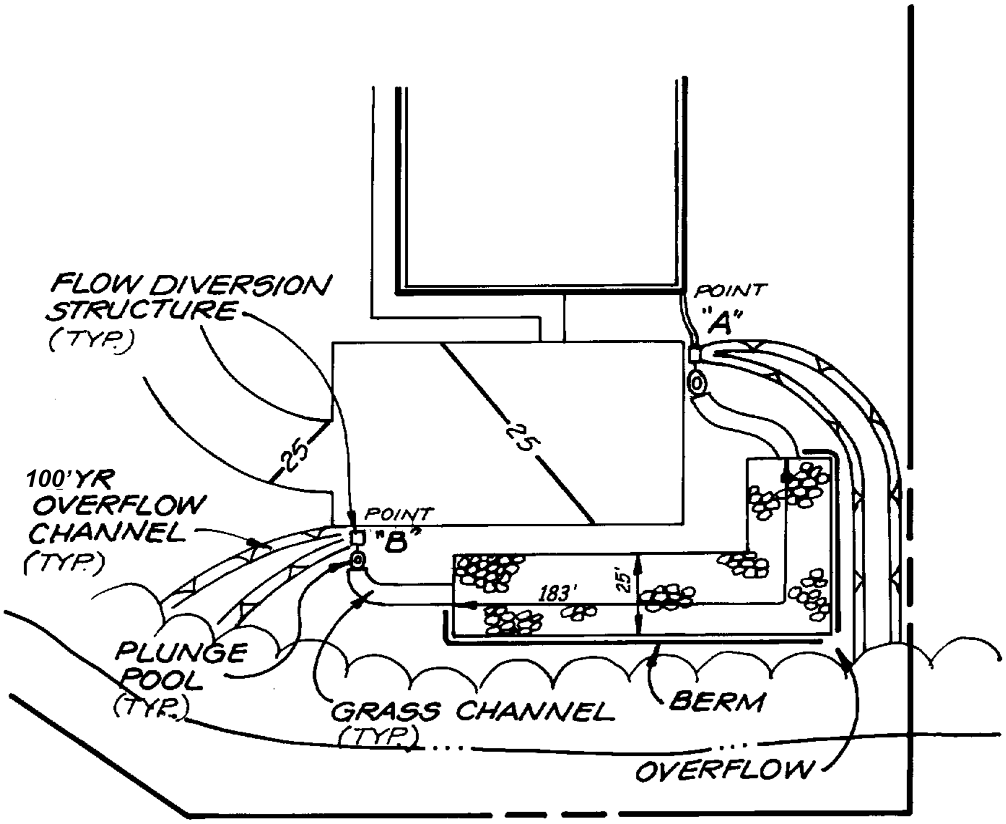


Figure 29.26 Infiltration Trench Site Plan

Size the second diversion (Point B) using the same techniques.

Peak flow equal 2/3 of the value for the entire site. Thus:

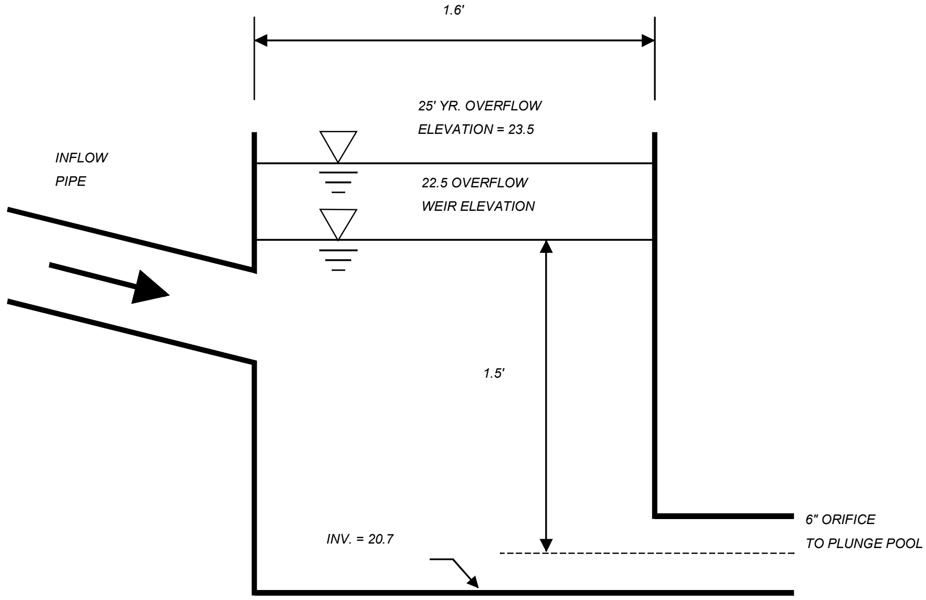
$Q_{100\text{-year}}$ = 29*0.67 = 19.3 cfs
Peak flow for WQ_v = 2.86*0.67 = 1.47 cfs

Size the low flow orifice to pass 1.47 cfs with 1.5' of head using the Orifice equation.

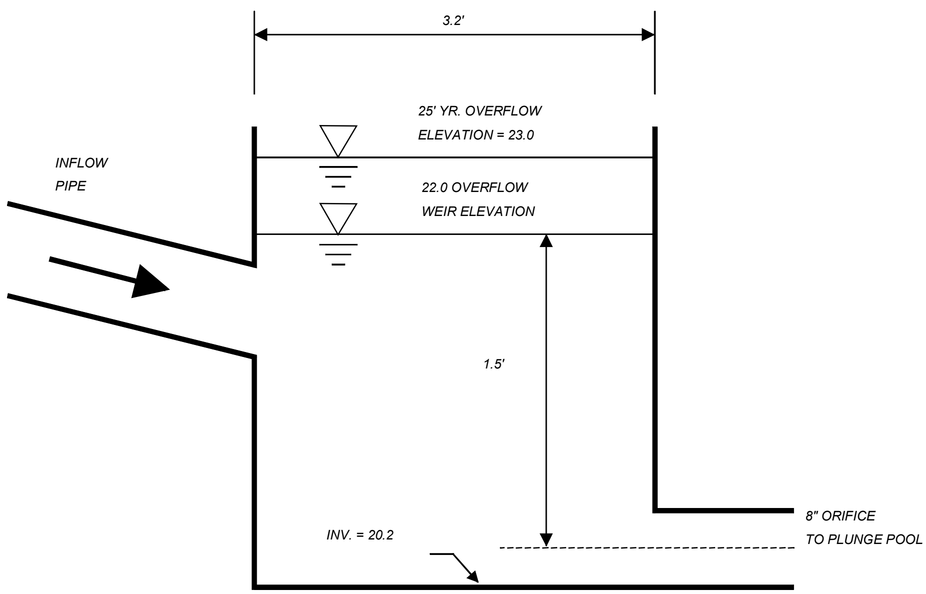
$Q=CA(2gh)^{½}$; 1.91 cfs = $0.6A(2 \times 32.2 \text{ ft/s}^2 \times 1.5')^{½}$
A=0.32 sq. ft. = $\pi d^2/4$; d = 0.64'; use 8" pipe with 8" gate valve

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; use 5.6' (see Figure 29.27)



POINT "A"



POINT "B"

Figure 29.27 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 $8{,}102 \times 0.25 = 2{,}026$ 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 29.28). 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 Pool*s

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>

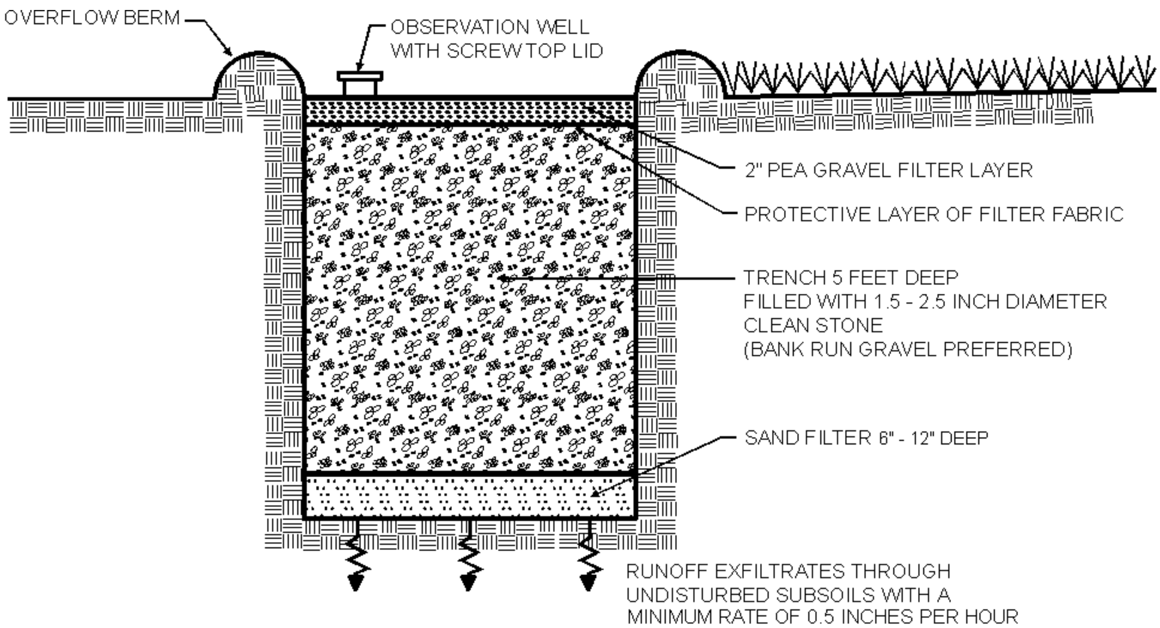


Figure 29.28 Infiltration Trench Cross Section

Step 8: Design Spillway(s)

Adequate stormwater 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.

29.6 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 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 29.29.

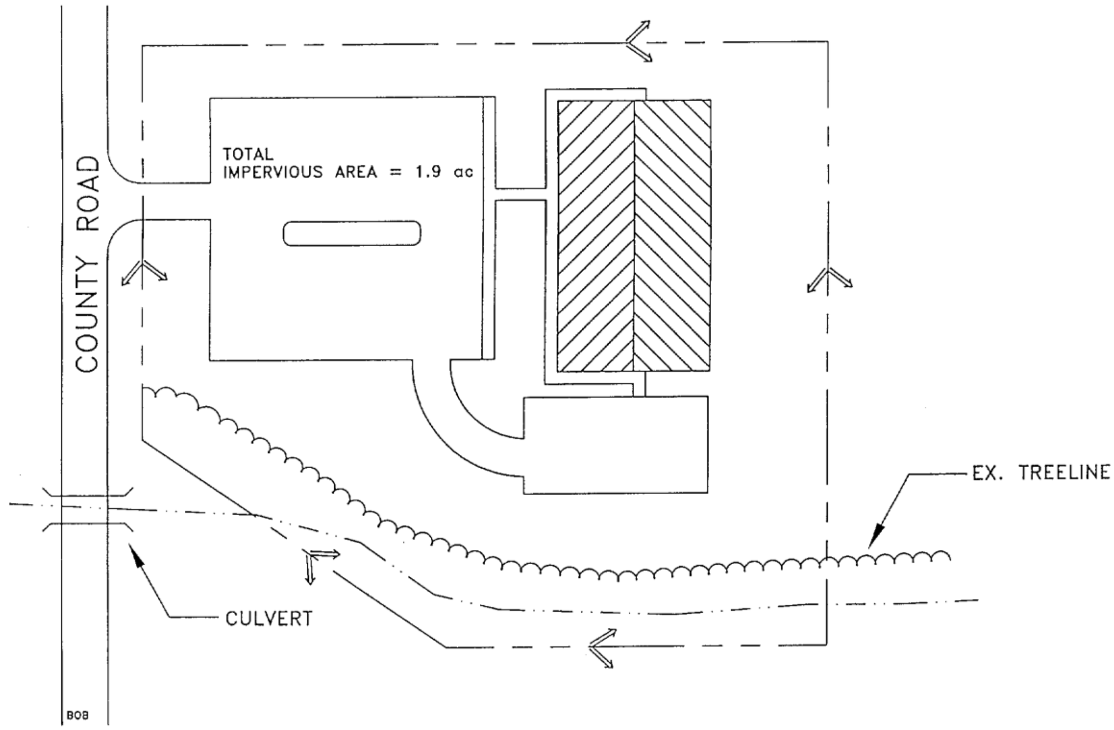


Figure 29.29 Wellington Recreation Center Site Plan

Base Data	Hydrologic Data		
		Pre	Post
Site Area = Total Drainage Area (A) = 3.0 ac			
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 Stormwater 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 Focus Areas

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 1.1 of the Hydrology Technical Manual* 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 29.30 and 29.31)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\text{ year}}$
		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 29.9 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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 29.30 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 1.6 Hydrology Section	CN from TABLE 1.9 Hydrology Section	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		77	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 29.31 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 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 29.32 for representative site plan).

Unit Peak Discharge (q_u), csm/in

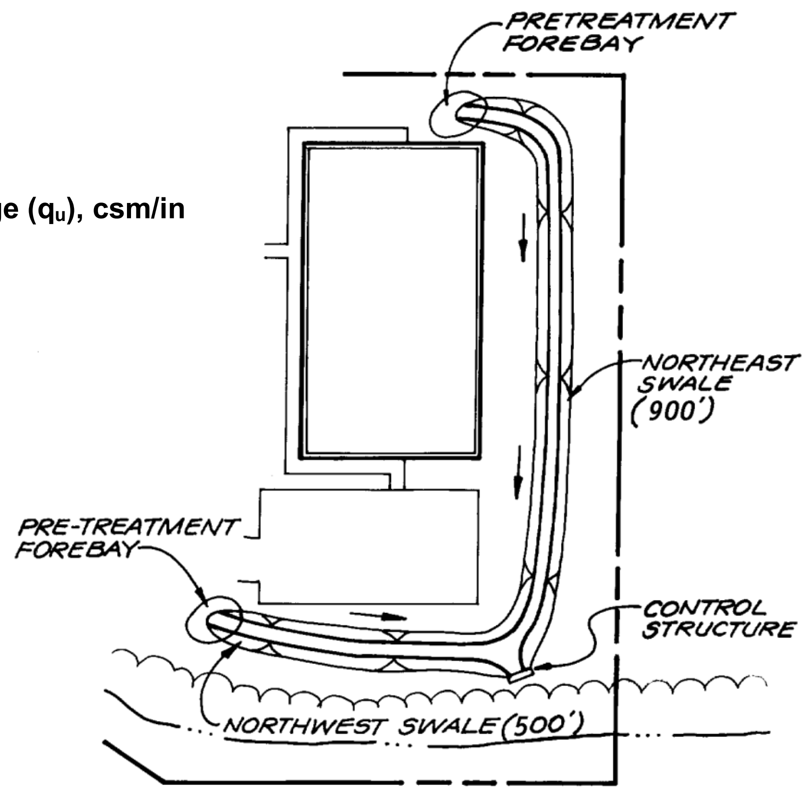


Figure 29.32 Enhanced Dry Swale Site Plan

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 29.5 and 29.6. 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 29.34).

(6.2 sq ft) (1,400 ft) = 8,680cubic feet [> WQ_v of 8,102 ft^3; OK]

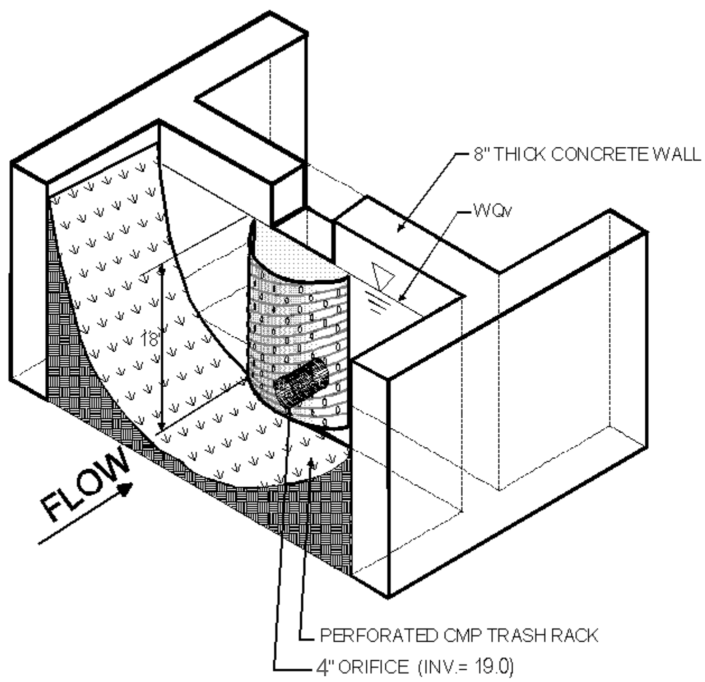


Figure 29.33 Control Structure at End of Swale

Step 6: Compute number of check dams (or similar structure) required to detain WQ_v (see Figure 29.7)

For the northwest fork, 500 ft @ 1.0% slope, and maximum depth at 18", place checkdams at:

1.5'/0.01 = 150' <u>place at 150', 4 required</u>

For the northeast fork, 900 ft @ 1.0% slope, and maximum 18" depth, place checkdams at:

1.5'/0.01 = 150' <u>place at 150', 6 required</u>

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 29.34).

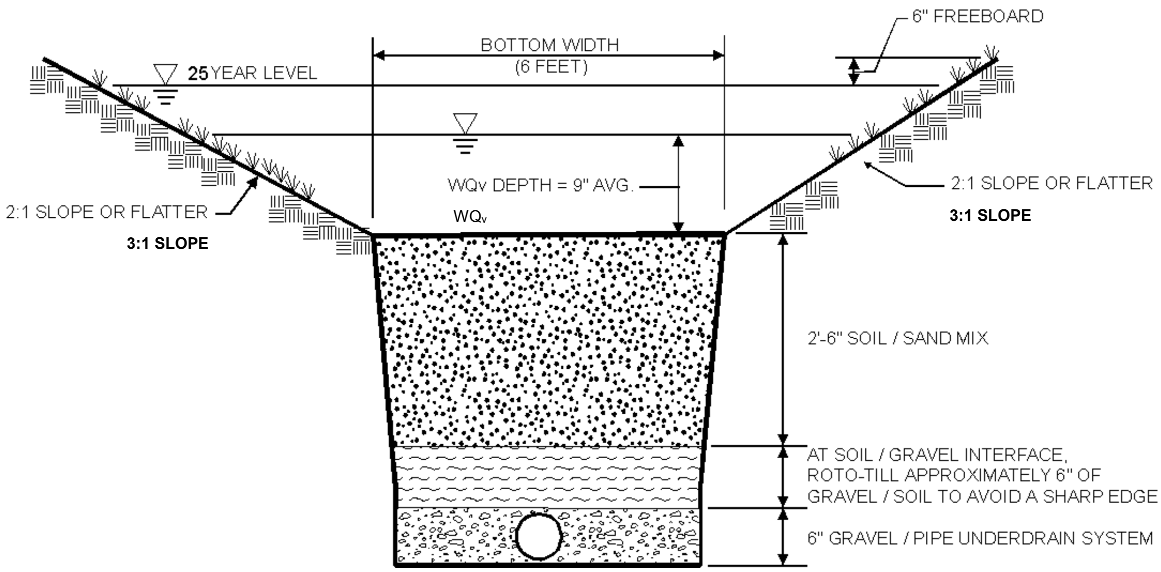


Figure 29.34 Trapezoidal Dry Swale Section

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:

$L = 23 \text{ cfs}/ (3.1*1.2^{1.5}) = 5.6'$ Use 5 ft

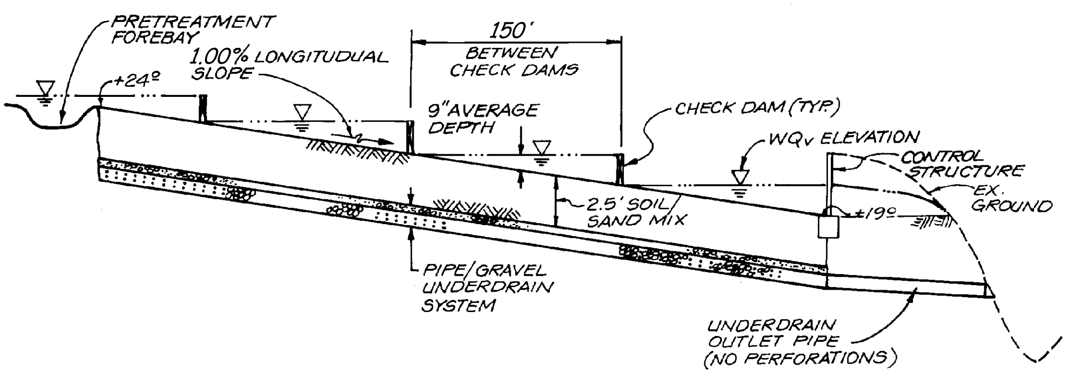


Figure 29.35 Profile of Northwest Fork Dry Swale

Step 9: Design low flow orifice at downstream headwall and checkdams (See Figure 29.33)

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 29.35

Step 11: Prepare Vegetation and Landscaping Plan

30.0 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.

Construction Controls:

1.0 Overview of Construction Controls
2.0 Erosion Controls
3.0 Sediment Controls
4.0 Material Waste Controls

Table of Contents

List of Tables

List of Figures

1.0 Overview of Construction Controls

1.1 Introduction

In order to address the requirements of pollution reduction at construction sites, a variety of controls should be employed to reduce soil erosion, reduce sediment loss from the site, and manage construction-generated waste and construction related toxic materials. Controls consist of both temporary and permanent methods to reduce pollution from a construction site.

The majority of controls address loss of soil from the site. Soil loss in the form of erosion and sediment due to storm events and wind constitute the majority of pollution generated from construction sites. Controls that address erosion and sediment are typically more site specific than waste and toxics management. Erosion and sediment controls are dependent on site slopes, drainage patterns and drainage quantities along with other site-specific conditions. Materials and waste management consists primarily of “good housekeeping” practices which are dependent on the type of construction and the quantity and type of building materials.

1.2 Control Selection Guide

The designer preparing the iSWM Construction Plan can first use the control selection guide on the following pages to determine the controls that are most appropriate for the site. Chapters 2.0, 3.0 and 4.0 contain the descriptions, design requirements, maintenance requirements, and limitations of the controls. These provide the tools for the designer to select the appropriate controls and properly locate them on the site, to effectively reduce erosion and sediment loss.

The Efficiency Ratings listed for the controls are the range of average efficiencies in reducing erosion or trapping sediment for the control, assuming the controls are properly designed, installed, and maintained for the flow and volumes from the design storm. The removal efficiency varies within in the range based on soil type.

The Efficiency Ratings are useful in comparing the effectiveness of the controls. The ratings are also used in calculating the Site Rating, which is used by some municipalities to ensure adequate design of erosion and sediment controls. Refer to *Section 1.3 Site Rating Calculations* for additional details concerning the Efficiency Ratings and the methodology for calculating the Site Rating.

The following legend applies to the Targeted Pollutants and Implementation Considerations presented for each of the controls:

Legend	
●	Significant Impact
◒	Medium Impact
○	Low Impact
?	Unknown or Questionable Impact

1.2.1 Erosion Controls

These controls are the measures and techniques used to retain soil in place. They are installed on the perimeter of the site to limit flow across disturbed areas and within the site to provide protective covering of disturbed areas that are not actively being worked. Erosion controls reduce the amount of soil removed and transported by stormwater runoff and reduce the need for sediment controls.

Table 1.1 Erosion Controls

Control	*Primary Purpose*	*Efficiency Rating (Fe)*
Check Dam	Slow flow to prevent erosion of swales and drainage ditches while also providing minor detention and sediment removal	0.30 - 0.50 *(Depends on soil type)*
Diversion Dike	Route flows around slopes and disturbed areas	0.95
Erosion Control Blankets	Protect disturbed soil and slopes from erosion using a degradable, rolled erosion control product; also provides limited protection as a perimeter control	*0.90 (Ground cover)* 0.65 *(Perimeter w/o vegetation)*
Interceptor Swale	Route flows around slopes and disturbed areas	0.95
Mulching	Protect disturbed soil with a layer of straw, wood chips, compost or other organic material	0.75 - 0.90 *(Depends on coverage)*
Pipe Slope Drain	Route overland flow on a slope into a pipe to protect the slope	0.95
Soil Surface Treatments	Protect disturbed soil from wind erosion (dust control) while also providing some protection from water erosion, depending on the treatment method	0.10 - 0.90 *(Depends on type of treatment)*
Turf Reinforcement Mats	Protect disturbed soil on steep slopes and in channels from erosion using a non-degradable, rolled erosion control product	0.90
Vegetation	Prevent erosion by providing a natural cover through hydro-mulching, seeding or sod placement	0.90 *(When fully established; lower while vegetation is first growing)*
Velocity Dissipation Devices	Protect soil from erosion at points where concentrated flows are discharged	N/A

> The Efficiency Ratings listed for the erosion controls are the assumed average efficiencies in reducing erosion, based on the controls being designed for the flow and volume from the temporary control design storm and installed in accordance with the criteria in this manual.

1.2.2 Sediment Controls

These controls are temporary structures or devices that capture soil transported by stormwater through sedimentation, filtration or chemical treatment of the runoff. They are used to trap sediment before it leaves the construction site. The effectiveness of controls that form a barrier or filter for trapping soil is highly dependent on the size of soil particles. The efficiencies presented are **ranges based on soil types**. The removal efficiency will be at the high end of the range for sand and coarse silt or loam and at the low end for fine silt or loam and clay. Controls with a single number for the efficiency rating do not vary in performance based on soil type.

Table 1.2 Sediment Controls		
Control	***Primary Purpose***	***Efficiency Rating (Fe)***
Active Treatment System	Remove pollutants and suspended soil, including fine clay particles, through filtration and/or chemical-aided flocculation	0.99
Depressed Grade Sediment Trap	Detain and settle suspended soil from small areas within rights-of-way	0.50 - 0.75
Dewatering Controls	Remove suspended soil from water that is pumped out of low points onsite	0.50 - 0.75
Inlet Protection	Intercept sediment at curb and area inlets as a secondary defense in sequence with other controls	0.35 - 0.65
Organic Filter Berm	Slow and filter runoff to retain sediment	0.50 - 0.75
Organic Filter Tubes	Slow and filter runoff to retain sediment	0.50 - 0.75
Passive Treatment System	Improve performance of other controls by adding flocculation agents to stormwater	0.85
Pipe Inlet Protection	Detain stormwater for sedimentation and filtration before it enters a closed conveyance system	0.50 - 0.75
Sediment Basin	Detain stormwater in a pond with a controlled outflow to allow for sedimentation	0.50 - 0.90
Silt Fence	Slow and filter runoff to retain sediment	0.50 - 0.75
Stabilized Construction Exit	Reduce offsite sediment tracking from trucks and construction equipment	N/A
Stone Outlet Sediment Trap	Intercept and filter small, concentrated flows in swales and other defined waterways	0.50 - 0.85
Triangular Sediment Filter Dike	Slow and filter runoff to retain sediment	0.50 - 0.75
Turbidity Barrier	Detain and settle suspended soil where work is occurring in or adjacent to a water body	0.50 - 0.90
Vegetated Filter Strips and Buffers	Slow sheet flow from small areas to allow for sedimentation	0.35 – 0.85 (*Depends on many conditions in addition to soil type*)
Wheel Cleaning Systems	Reduce offsite sediment tracking from trucks and construction equipment	N/A

The Efficiency Ratings listed for the sediment controls are the assumed average efficiencies in capturing sediment for a range of soil types, based on the controls being designed for the flow and volume from the temporary control design storm and installed in accordance with the criteria in this manual.

1.2.3 Material and Waste Controls

Material and waste control techniques are applicable on the majority of construction projects due to their general purpose of reducing the discharge of pollutants from construction activities. They form the basis of good housekeeping procedures that should be followed during construction and in many cases are mandated by stormwater discharge permits. The techniques are essential to preventing the discharge of pollutants other than sediment from a construction site.

A numeric efficiency rating is not provided for material and waste controls, since the controls are not for erosion and sediment and are not a factor in the Site Rating calculation. All of these techniques are highly effective in minimizing discharges of the targeted pollutants when properly applied.

Table 1.3 Material and Waste Controls

Control	*Primary Purpose*
Chemical Management	Techniques to minimize the exposure of paints, solvents, fertilizer, pesticides, herbicides, and other chemicals to precipitation and stormwater; and techniques for managing the wastewater from washout of paint, form release oils, curing compounds, and other construction chemicals
Concrete Sawcutting Waste Management	Techniques for collection and disposal of the slurry of cutting water and concrete cuttings that results from concrete sawing
Concrete Waste Management	Techniques for disposal of concrete washout, demolished concrete, etc.
Debris and Trash Management	Techniques for storage and disposal of packaging, scrap building materials, personal trash, and other wastes generated by construction activities and personnel
Hyper-Chlorinated Water Management	Techniques to prevent water with high concentrations of chlorine from being discharged
Sandblasting Waste Management	Techniques for disposal of sandblasting waste and containment of wastes during operations
Sanitary Waste Management	Techniques to control and prevent the exposure of sanitary waste to precipitation and stormwater
Spill and Leak Response Procedures	Techniques to minimize the discharge of pollutants from spills and leaks
Subgrade Stabilization Management	Techniques to control runoff from soil being chemically stabilized in preparation for construction
Vehicle and Equipment Management	Techniques to prevent discharges of fluids used in vehicle and equipment operation and maintenance and the discharge of wash waters that contain soaps or solvents

1.3 Site Rating Calculation

1.3.1 Introduction

The site rating calculation is a useful tool for evaluating the potential effectiveness of proposed erosion and sediment controls on a construction site. It is used to compare the potential soil loss from a site without controls to the soil loss from the site with proposed controls. The site rating may also be used to compare the effectiveness of two different controls on a site.

The site rating calculation is an optional element for an iSWM Construction Plan but may be required by some municipalities in North Central Texas. When required, a numeric site rating is established as the criteria for the design of erosion and sediment controls for a construction site. Municipalities that use the site rating will typically require a minimum site rating of 0.70, which reflects a realistic, attainable reduction in sediment loss from a construction site of 70 percent using controls compared to the same site without the use of controls.

The user of this manual is advised to confirm local requirements with the municipality where the project is located. When required to provide the site rating by the local government, the iSWM Construction Plan should be prepared as described in Chapter 4 of the iSWM Criteria Manual, followed by calculation of the site rating. Controls shall then be modified and added as needed to achieve the minimum required site rating.

1.3.2 Background

The design and implementation of erosion and sediment controls is highly dependent on project site conditions and construction methods. The amount of potential soil loss from a site is based on the physical features and location of the site: soil type(s), slope, length of stormwater flow across the site, the rainfall intensity and overall runoff quantity of a particular storm, and the groundcover of the site. Of these factors, construction activity at a site can affect the groundcover, the slope of the site and the length of stormwater flow across the site. These effects are mitigated by minimizing onsite disturbance of the soil and groundcover and providing structural measures to retain sediment onsite after erosion occurs.

The most effective method to reduce sediment loss from a tract of land is to prevent the occurrence of erosion. While structural barriers, such as those shown in this manual, have a theoretical 70 to 90 percent effectiveness rating for removal of sediment from runoff, natural groundcover and mulching can provide up to 98 percent reduction in erosion and site soil loss. Therefore, the primary goals of the erosion control plan for a construction site is to prevent the soil from eroding and to minimize the area of disturbance through the phasing of construction activities, mulching of disturbed but inactive areas, and providing tarps, seeding or hydromulching of stockpiles. These techniques are not only the most effective at reducing soil loss; they are normally the most cost effective due to low initial cost and reduced maintenance requirements.

Sediment removal controls provide the second line of defense by treating sediment-laden stormwater before it is discharged from the site. All construction activities will require areas in which soil is disturbed. Stormwater runoff that crosses areas of exposed soil will require treatment by adequate Best Management Practices in accordance with the guidelines presented in this manual. Sediment removal controls include diversion of stormwater around areas of construction, and filtration and sedimentation (detention) of sediment-laden runoff that crosses disturbed areas.

1.3.3 Methodology

Site Rating Description

The runoff across both disturbed and non-disturbed areas of a drainage basin produces a quantity of soil loss due to erosion. This quantity is estimated through the use of the Universal Soil Loss Equation as a mass per time period. Erosion and sediment controls are used to reduce the sediment transported offsite.

The site rating is defined as the theoretical amount of soil that remains uneroded and/or is captured on a site through the use of erosion and sediment controls (soil retained) divided by the theoretical amount of soil that would leave the site if no controls were used (uncontrolled). A minimum site rating of 0.70 is typically used as a guideline for the adequate design of erosion and sediment control systems.

This **site rating** is calculated as follows:

$$SR = ZA_{retained} / ZA_{uncontrolled} \qquad (1.1)$$

where:

SR = Site Rating

$ZA_{retained}$ = Soil uneroded and/or retained onsite by erosion prevention and sediment trapping practices (pounds/year)

$ZA_{uncontrolled}$ = Soil loss from site if no controls used (pounds/year)

Note that the site rating calculation methodology assumes that the erosion and sediment control measures are correctly designed, installed, and maintained in accordance with the criteria in this manual to treat the volume of runoff from the 2-year, 24-hour storm event, which is the regionally defined design storm frequency for temporary control design.

Universal Soil Loss Equation

Several elements are involved in evaluating the potential for erosion of a site. Soil type, length of flow across the ground, slope of ground, rainfall intensity and groundcover play important roles in determining if a site will produce excessive siltation downstream. The Universal Soil Loss Equation is used to determine the potential erodibility of a site. The Universal Soil Loss Equation (USLE) is expressed as:

$$Z = R * K * LS * C_s * P \qquad (1.2)$$

where:

Z = Rate of soil loss (tons per acre per year)

R = Rainfall erosion factor (300 for North Texas)

K = Soil erodibility factor

LS = Length/slope factor

C_s = Cropping/management factor

P = Erosion control practice

Calculate the anticipated yearly soil loss (ZA)

$$ZA = Z * A \qquad (1.3)$$

where

ZA = Soil loss per year (tons per year)

Z = Rate of soil loss for a drainage basin (tons per acre per year)

A = Area of drainage basin (acres)

Some of the factors above (R and K) remain constant throughout the construction of the project. Both the LS and C_s factors are altered during construction through clearing, grading and drainage operations on the site. The P factor represents the implementation of erosion and sediment controls to reduce the potential for sediment to be transported offsite. These factors are discussed in the following sections.

Rainfall Erosion Factor

The average annual rainfall erosion factor, R, varies for different regions throughout the country and during the year. This value accounts for the volume and intensity of rainfall for a one year time period in a region. A value of 300 is used for R in the North Central Texas area.

Soil Erodibility Factor

The soil erodibility factor, K, indicates the potential for water erosion of the soil. It is strongly suggested that soil erodibility be determined as part of the geotechnical investigation of the site in order to determine the most effective means to reduce site erosion. If a site has not been previously disturbed, the native soil type(s) most likely to be present at the site can be identified on the NRCS Web Soil Survey at: http://websoilsurvey.nrcs.gov/app/. The website also contains the soil erodibility factors for native soils.

Consider the depth of grading activities when determining the soil erodibility factor. Soil type varies with depth. The surface soil may have a low erodibility factor, but the soil at a lower depth may have a high erodiblility factor when it is exposed by grading operations.

Table 1.4 provides approximate values of K for various soil types and can be used in calculations if detailed data are not available.

Table 1.4 Soil Erodibility Factors (K)*

Soil Type	*K*
Sand	0.03
Fine Sand	0.14
Loamy Sand	0.10
Sandy Loam	0.24
Loam	0.34
Silt Loam	0.42
Silt	0.52
Sandy Clay Loam	0.25
Clay Loam	0.25
Silty Clay Loam	0.32
Sandy Clay	0.13
Silty Clay	0.23
Clay	0.13 – 0.29

(Source: Standard Handbook of Environmental Engineering edited by Robert A. Corbitt)
*Assuming 2% organic matter content.

Length/Slope Factor

The length-slope factor, LS, of the drainage basin may be changed through construction operations. A reduction in slope or drainage length can significantly reduce the erosion potential of the drainage basin. The length-slope factor considers the topographic features of the drainage basin. The LS factor is defined by the length and slope that a drop of water will travel through the drainage basin from the farthest reach to the point of analysis. The slope value is the average slope of this path. Table 1.5 lists values of LS for a wide variety of slope and drainage length. LS can also be calculated as follows:

$$\mathbf{LS = [L/72.6]^{M} * [65.41*\sin^2(S) + 4.56*\sin(S) + 0.065]} \qquad \mathbf{(1.4)}$$

where:

L = Length of flow path of contributing area (feet)

M = 0.6 * [1 - exp(-35.835*s)] where s=slope (feet/feet)

S = Average slope of contributing area (degrees)

Table 1.5 Length/Slope Factor (LS)

Length (ft.)	Slope (ft/ft)												
	0.005	**0.01**	**0.015**	**0.02**	**0.025**	**0.03**	**0.04**	**0.05**	**0.06**	**0.1**	**0.15**	**0.2**	**0.3**
10	0.07	0.08	0.09	0.10	0.11	0.12	0.14	0.17	0.20	0.37	0.67	1.06	2.06
20	0.08	0.09	0.11	0.12	0.14	0.16	0.20	0.24	0.29	0.55	1.01	1.60	3.13
30	0.08	0.10	0.12	0.14	0.16	0.18	0.23	0.29	0.36	0.70	1.29	2.05	3.99
40	0.08	0.11	0.13	0.15	0.18	0.21	0.27	0.34	0.42	0.82	1.53	2.43	4.74
50	0.09	0.11	0.13	0.16	0.19	0.22	0.30	0.38	0.47	0.94	1.75	2.78	5.42
60	0.09	0.11	0.14	0.17	0.21	0.24	0.32	0.41	0.52	1.04	1.95	3.10	6.04
70	0.09	0.12	0.15	0.18	0.22	0.26	0.35	0.45	0.56	1.14	2.13	3.40	6.63
80	0.09	0.12	0.15	0.19	0.23	0.27	0.37	0.48	0.60	1.23	2.31	3.68	7.18
90	0.09	0.12	0.16	0.19	0.24	0.28	0.39	0.51	0.64	1.32	2.48	3.95	7.71
100	0.09	0.12	0.16	0.20	0.25	0.30	0.41	0.53	0.68	1.41	2.64	4.21	8.21
125	0.09	0.13	0.17	0.22	0.27	0.32	0.45	0.60	0.76	1.60	3.02	4.81	9.39
150	0.10	0.13	0.18	0.23	0.28	0.35	0.49	0.66	0.84	1.78	3.36	5.37	10.47
175	0.10	0.14	0.18	0.24	0.30	0.37	0.53	0.71	0.91	1.95	3.69	5.89	11.49
200	0.10	0.14	0.19	0.25	0.32	0.39	0.56	0.76	0.98	2.11	3.99	6.38	12.45
250	0.10	0.15	0.20	0.27	0.34	0.42	0.62	0.85	1.10	2.40	4.56	7.29	14.23
300	0.10	0.15	0.21	0.28	0.36	0.46	0.67	0.93	1.22	2.67	5.09	8.14	15.87
350	0.10	0.16	0.22	0.30	0.38	0.49	0.72	1.00	1.32	2.92	5.58	8.92	17.41
400	0.11	0.16	0.23	0.31	0.40	0.51	0.77	1.07	1.42	3.16	6.04	9.67	18.86
450	0.11	0.16	0.23	0.32	0.42	0.54	0.81	1.13	1.51	3.38	6.48	10.37	20.25
500	0.11	0.17	0.24	0.33	0.44	0.56	0.85	1.20	1.59	3.59	6.90	11.05	21.57
600	0.11	0.17	0.25	0.35	0.47	0.60	0.92	1.31	1.75	4.00	7.70	12.33	24.06
700	0.11	0.18	0.26	0.37	0.49	0.64	0.99	1.42	1.90	4.37	8.44	13.52	26.39
800	0.11	0.18	0.27	0.38	0.52	0.67	1.05	1.51	2.04	4.73	9.14	14.65	28.59
900	0.11	0.18	0.28	0.39	0.54	0.70	1.11	1.60	2.18	5.07	9.81	15.72	30.69
1000	0.12	0.19	0.28	0.41	0.56	0.73	1.17	1.69	2.30	5.39	10.44	16.74	32.69
1500	0.12	0.20	0.32	0.46	0.64	0.86	1.40	2.07	2.85	6.82	13.31	21.35	41.69
2000	0.12	0.21	0.34	0.50	0.71	0.97	1.60	2.39	3.32	8.07	15.80	25.37	49.55
3000	0.13	0.23	0.37	0.57	0.82	1.13	1.93	2.93	4.12	10.22	20.13	32.35	63.19
4000	0.13	0.24	0.40	0.62	0.91	1.27	2.20	3.38	4.80	12.09	23.90	38.44	75.10
5000	0.14	0.25	0.43	0.67	0.99	1.39	2.43	3.78	5.40	13.77	27.31	43.95	85.86

Cropping/Management Factor

The cropping factor, C_s, considers the protection of natural ground cover in preventing erosion of the soil. This is dependent on the type of vegetation (grass or trees) and the density of the vegetation on the site. The higher the value for C, the less protection from erosion is available; for example, a bare construction site with no groundcover has a C value of 1.0, while hay mulch applied at 1 ton per acre produces a C value of 0.13.

The C_s factor is not intended to account for the reduced erosion provided by temporary or final vegetation established on areas that have been disturbed. The erosion control factor, P, described below reflects the erosion protection afforded by use of vegetation in accordance with the *Section 2.9 Vegetation*.

Table 1.6 provides approximate values for C_s for a variety of conditions. The sensitivity of the C_s value reflects the importance of minimizing the area of disturbance and providing protection to the disturbed soil before erosion occurs. **For existing bare areas or areas stripped of natural vegetation by construction, a C_s value of 1.0 shall be used.**

Table 1.6 Cropping Factors							
Type and Height of Raised Vegetative Canopy	*Canopy Cover, %*	*Ground cover that contacts the surface, %*					
		0	**20**	**40**	**60**	**80**	**95-100**
No appreciable canopy / Canopy of tall weeds or short brush (<1' tall)	0	0.450	0.200	0.100	0.042	0.013	0.003
	25	0.360	0.170	0.090	0.038	0.012	0.003
	50	0.260	0.130	0.070	0.035	0.012	0.003
	75	0.170	0.100	0.060	0.031	0.011	0.003
Appreciable brush or bushes (5' fall height)	25	0.400	0.180	0.090	0.040	0.013	0.003
	50	0.340	0.160	0.085	0.038	0.012	0.003
	75	0.280	0.140	0.080	0.036	0.012	0.003
Trees w/o appreciable low brush (>10' fall height)	25	0.420	0.190	0.100	0.041	0.013	0.003
	50	0.390	0.180	0.090	0.040	0.013	0.003
	75	0.360	0.170	0.090	0.039	0.012	0.003

(Source: Standard Handbook of Environmental Engineering edited by Robert A. Corbitt)

For each drainage basin, this C_s value is weighted based on the percentage of disturbed area in the basin:

$$\mathbf{C_{stotal} = [(C_{sun}*A_{un}) + (C_{sdis}*A_{dis})] / A_{total}} \qquad \textbf{(1.5)}$$

where:
C_{stotal} = C_s for drainage basin
C_{sun} = C_s for undisturbed areas
A_{un} = Area of undisturbed areas of drainage basin (acres)
C_{sdis} = C_s for disturbed areas
A_{dis} = Area of disturbed areas of drainage basin (acres)
A_{total} = Total area of drainage basin (acres)

Erosion Control Practice Factor

The erosion control practice factor, P, accounts for the erosion control and sediment trapping effectiveness of land treatments such as mulching, erosion control blankets, temporary or final vegetation, sediment basins, filter berms, check dams, and other controls.

For the, A P value of 1 is used in the USLE calculation of the uncontrolled soil loss from the site ($ZA_{uncontrolled}$) because it is assumed that no controls are used.

The Efficiency Rating (F_e) for the calculation of the soil erosion prevented/sediment retained on the site ($ZA_{retained}$) for the various controls is used in place of the erosion control practice factor. The Efficiency Rating is the compliment of the P value (F_e = 1 - P) and is used instead of P, because the desired calculation is the soil retained on the site through the use of the practices rather than the soil lost from the site.

When multiple structural controls are used in series to treat runoff from disturbed areas, the design efficiency can be calculated as follows[1]:

$$\mathbf{F_{eTOTAL} = 1-((1-F_{e1})*(1-F_{e2}))} \qquad \textbf{(1.6)}$$

where:

F_{e1} = Removal efficiency of first control

F_{e2} = Removal efficiency of second control

Site Rating Factor Calculation

After erosion potential is calculated for both uncontrolled ($ZA_{uncontrolled}$) and controlled conditions ($ZA_{retained}$), a site rating (SR) is calculated using Equation 1.1.

A minimum design storm of 2-year intensity and 24-hour duration shall be used for design of structural sediment control techniques. Other design criteria are defined in sections of the manual for specific erosion controls. The 2-year intensity is the rainfall intensity that has a 50 percent probability of occurring in any given year. The 24-hour duration establishes the overall volume of rainfall and runoff of the storm with a peak flow of the referenced intensity. Municipalities can adjust this requirement for particularly sensitive areas or other areas of concern.

1.3.4 Summary

The following outlines the primary steps required to calculate the Site Rating.

I. Develop design storm flows.

Determine drainage sub-basin.
Determine C_s values and drainage patterns (LS) based on conditions for design period.

II. Calculate theoretical soil loss for each sub-basin if no controls are used.

Use value of 1 for the Erosion Control Practice factor, P, since no controls are used.

III. Calculate theoretical soil uneroded and/or retained for each sub-basin by use of controls.

Use F_e from *Section 2.0 Erosion Controls* and *Section 3.0 Sediment Controls* (or test/manufacturer's data) in place of P in USLE.

Calculate soil retained onsite due to use of controls.

IV. Determine site rating.

[1] *Hartigan, P. and K. Wilweding, The Clean Colorado Project and Urban Nonpoint Source Pollution Control: The LCRA Program, Seminar Publication - Nonpoint Source Watershed Workshop, Environmental Protection Agency, Sept. 1991, p. 170.*

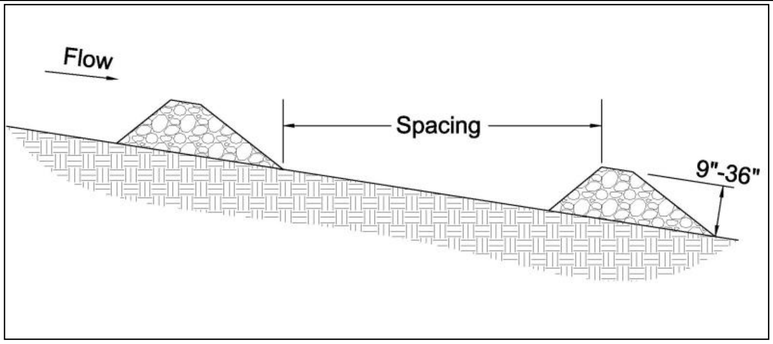
Total sediment loss from the site must be reduced by a minimum of 70 percent from uncontrolled conditions (Site Rating > 0.70).

For sites that include phasing of the construction, repeat the steps for each phase.

2.0 Erosion Controls

2.1 Check Dam

Erosion Control



Description: Check dams are small barriers consisting of loose rock, rock bags, or organic filter tubes placed across a drainage swale or ditch. They reduce the velocity of small concentrated flows, provide a limited barrier for sediment and reduce the potential for erosion of the swale or ditch.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Heights between 9 inches and 36 inches
- Top of the downstream dam should be at the same elevation as the toe of the upstream dam

ADVANTAGES / BENEFITS:

- Reduced velocities in long drainage swales or ditches
- May be used with other channel protection measures
- Provides some sediment removal

DISADVANTAGES / LIMITATIONS:

- Cannot be used in live stream channels
- Minor ponding upstream of the check dams
- Extensive maintenance or replacement of the dams required after heavy flows or high velocity flows
- Mowing hazard from loose rocks if all rock is not removed at end of construction

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove silt when it reaches approximately ⅓ the height of the dam or 12 inches, whichever is less

TARGETED POLLUTANTS

◒ Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
◒ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.30-0.50
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
● Maintenance
○ Training
◒ Suitability for Slopes > 5%

Other Considerations:

- *None*

2.1.1 Primary Use

Check dams are used in long drainage swales or ditches to reduce erosive velocities. They are typically used in conjunction with other channel protection techniques such as vegetation lining and turf reinforcement mats. Check dams provide limited treatment to sediment-laden flows. They are more useful in reducing flow velocities to acceptable levels for stabilization methods. Check dams may be used in combination with stone outlet sediment traps, where the check dams prevent erosion of the swale while the sediment trap captures sediment at the downstream end of the swale.

2.1.2 Applications

Check dams are typically used in swales and drainage ditches along linear projects such as roadways. They can also be used in short swales down a steep slope, such as swales down a highway embankment, to reduce velocities. Check dams shall not be used in live stream channels.

Check dams should be installed before the contributing drainage area is disturbed, so as to mitigate the effects on the swale from the increase in runoff. If the swale itself is graded as part of the construction activities, check dams are installed immediately upon completion of grading to control velocities in the swale until stabilization is completed.

2.1.3 Design Criteria

General Criteria

- Typically, the dam height should be between 9 inches and 36 inches, depending on the material of which they are made. The height of the check dam shall always be less than one-third the depth of the channel.
- Dams should be spaced such that the top of the downstream dam is at the same elevation as the toe of the upstream dam. On channel grades flatter than 0.4 percent, check dams should be placed at a distance that allows small pools to form between each check dam.
- The top of the side of the check dam shall be a minimum of 12 inches higher than the middle of the dam. In addition, the side of the dams shall be embedded a minimum of 18 inches into the side of the drainage ditch, swale or channel to minimize the potential for flows to erode around the side of the dam.
- Larger flows (greater than 2-year, 24-hour design storm) must pass the check dam without causing excessive upstream flooding.
- Check dams should be used in conjunction with other sediment reduction techniques prior to releasing flow offsite.
- Use geotextile filter fabric under check dams of 12 inches in height or greater. The fabric shall meet the following minimum criteria:
  - Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles, 250-lbs.
  - Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 135-lbs.
  - Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 420-psi.
  - Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 20 (max).
- Loose, unconfined soil, wood chips, compost, and other material that can float or be transported by runoff shall not be used to construct check dams.

Rock Check Dams

- Stone shall be well graded with stone size ranging from 3 to 6 inches in diameter for a check dam height of 24 inches or less. The stone size range for check dams greater than 24 inches is 4 to 8 inches in diameter.
- Rock check dams shall have a minimum top width of 2 feet with side slopes of 2:1 or flatter.

Rock Bag Check Dams

- Rock bag check dams should have a minimum top width of 16 inches.
- Bag length shall be 24 inches to 30 inches, width shall be 16 inches to 18 inches and thickness shall be 6 inches to 8 inches and having a minimum weight of 40 pounds.
- Minimum rock bag dam height of 12 inches would consist of one row of bags stacked on top of two rows of bag. The dam shall always be one more row wide than it is high, stacked pyramid fashion.
- Bags should be filled with pea gravel, filter stone, or aggregate that is clean and free of deleterious material.
- Sand bags shall not be used for check dams, due to their propensity to break and release sand that is transported by the concentrated flow in the drainage swale or ditch.
- Bag material shall be polypropylene, polyethylene, polyamide or cotton burlap woven fabric, minimum unit weight 4-ounces-per-square-yard, Mullen burst strength exceeding 300-psi as determined by ASTM D3786, Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, and ultraviolet stability exceeding 70 percent.
- PVC pipes may be installed through the dam to allow for controlled flow through the dam. Pipe should be schedule 40 or heavier polyvinyl chloride (PVC) having a nominal internal diameter of 2 inches.

Sack Gabion Check Dams

- Sack gabion check dams may be used in channels with a contributing drainage area of 5 acres or less.
- Sack gabions shall be wrapped in galvanized steel, woven wire mesh. The wire shall be 20 gauge with 1 inch diameter, hexagonal openings.
- Wire mesh shall be one piece, wrapped around the rock, and secured to itself on the downstream side using wire ties or hog rings.
- Sack gabions shall be staked with ¾ inch rebar at a maximum spacing of three feet. Each wire sack shall have a minimum of two stakes.
- Stone shall be well graded with a minimum size range from 3 to 6 inches in diameter.

Organic Filter Tube Check Dams

- Organic filter tubes may be used as check dams in channels with a contributing drainage area of 5 acres or less.
- Organic filter tubes shall be a minimum of 12 inches in diameter.
- Filter material used within tubes to construct check dams shall be limited to coir, straw, aspen fiber and other organic material with high cellulose content. The material should be slow to decay or leach nutrients in standing water.
- Staking of filter tubes shall be at a maximum of 4 foot spacing and shall alternate through the tube and on the downstream face of the tube.
- Unless superseded by requirements in this section, filter tubes and filter material shall comply with the

criteria in *Section 3.6 Organic Filter Tubes*.

2.1.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.9 Check Dam (Rock). Specifications are also available in the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT 2004), Item 506.2.A and Item 506.4.C.1.

2.1.5 Inspection and Maintenance Requirements

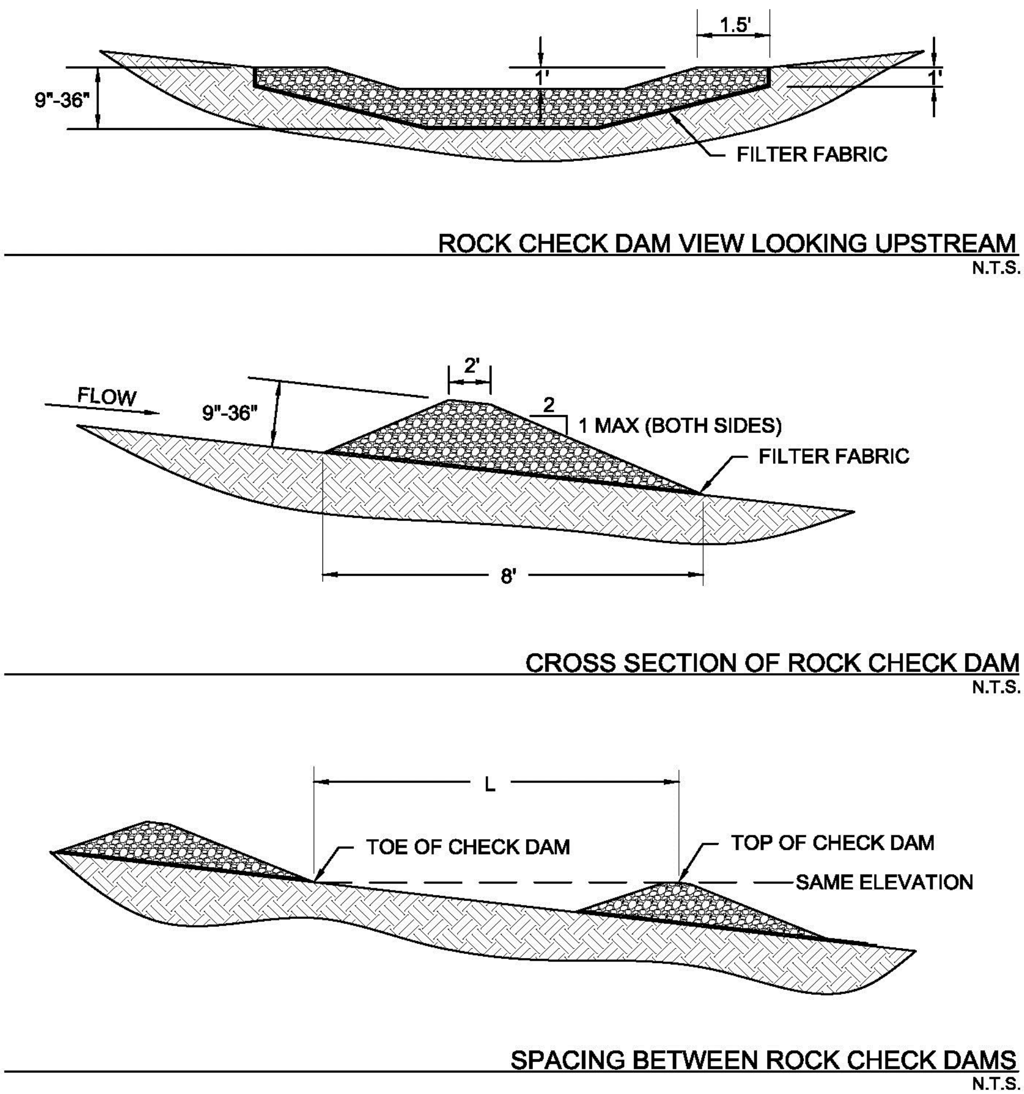
Check dams should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Silt must be removed when it reaches approximately 1/3 the height of the dam or 12 inches, whichever is less. Inspectors should monitor the edges of the dam where it meets the sides of the drainage ditch, swale or channel for evidence of erosion due to bypass or high flows. Eroded areas shall be repaired. If erosion continues to be a problem, modifications to the check dam or additional controls are needed.

Care must be used when taking out rock check dams in order to remove as much rock as possible. Loose rock can create an extreme hazard during mowing operations once the area has been stabilized.

2.1.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

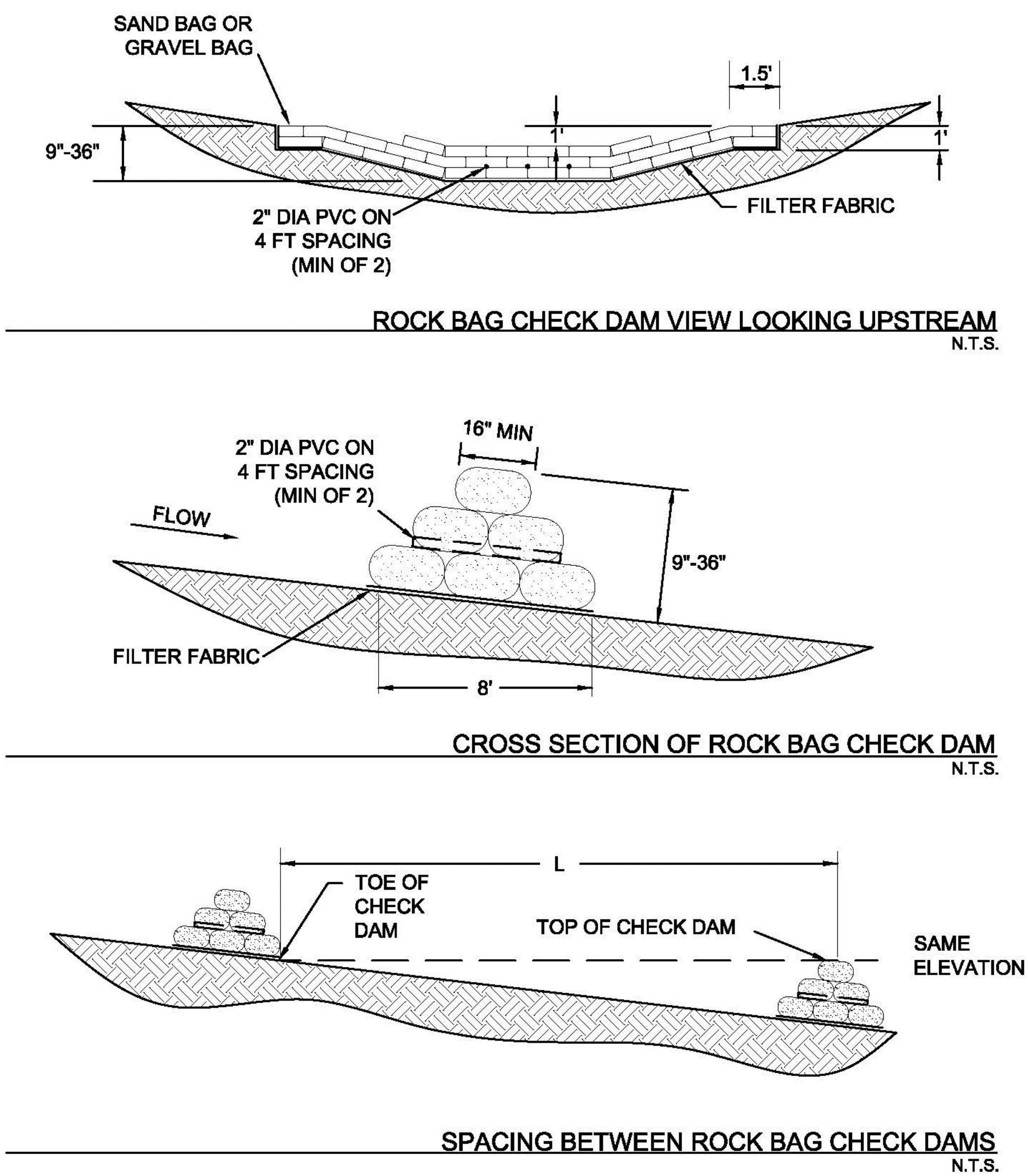
The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be adapted for the site by the designer. Dimensions and notes appropriate for the application must also be added by the designer.



NOTES: ACTUAL DIMENSIONS OF THE CHECK DAMS SHALL BE DESIGNED BASED ON FLOW CONDITIONS IN THE DRAINAGE SWALE OR DITCH. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE CHECK DAMS.

- HEIGHT OF CHECK DAMS BASED ON SWALE OR DITCH DIMENSIONS AND FLOW CONDITIONS.
- SPACING OF CHECK DAMS BASED ON GRADE OF THE SWALE OR DITCH. TOP OF DOWNSTREAM DAM SHALL BE AT SAME ELEVATION AS TOE OF UPSTREAM DAM

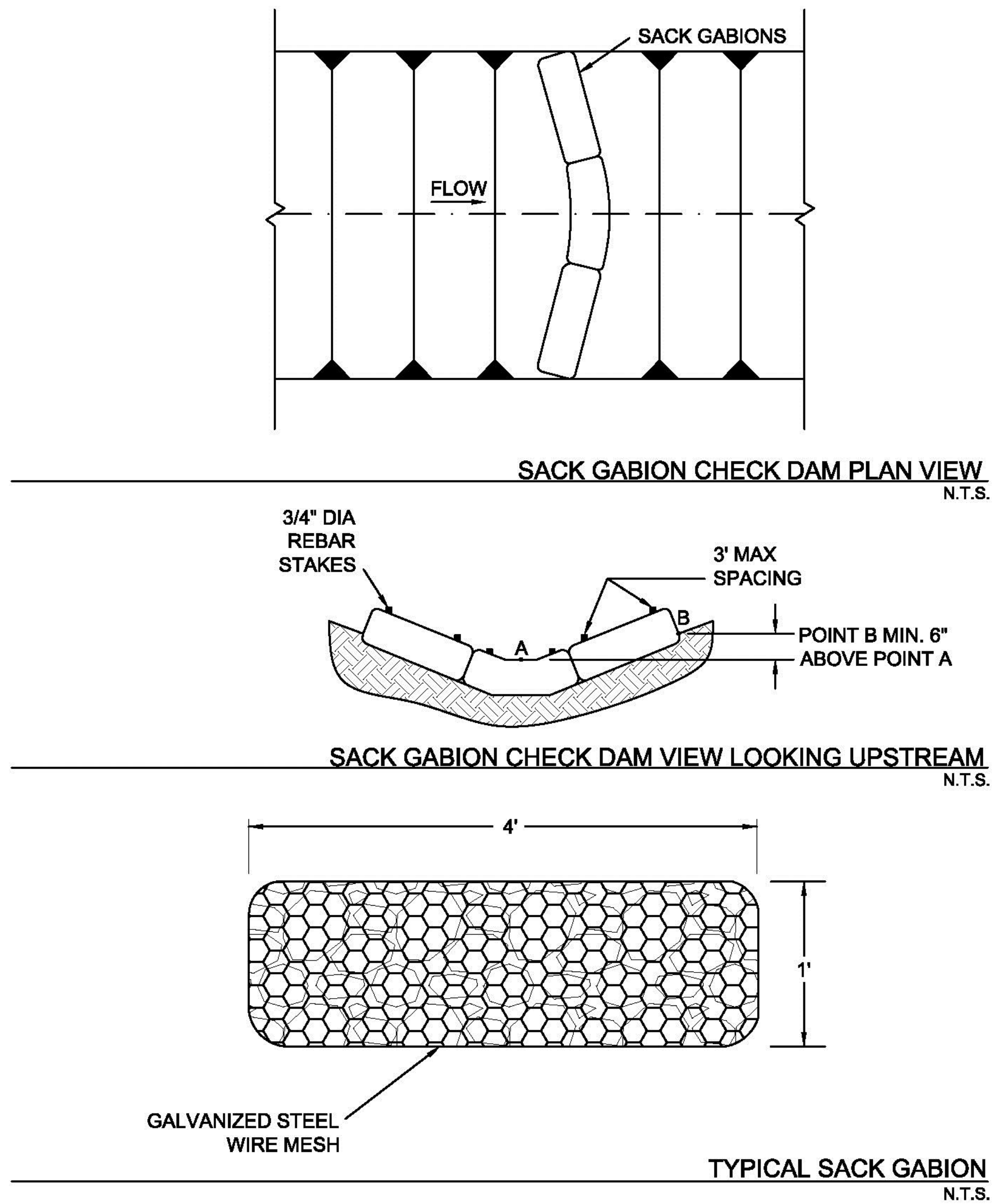
Figure 2.1 Schematics of Rock Check Dams
(Source: Modified from Stormwater Management Manual for Western Washington)



NOTES: ACTUAL DIMENSIONS OF THE CHECK DAMS SHALL BE DESIGNED BASED ON FLOW CONDITIONS IN THE DRAINAGE SWALE OR DITCH. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE CHECK DAMS.

- HEIGHT OF CHECK DAMS BASED ON SWALE OR DITCH DIMENSIONS AND FLOW CONDITIONS.
- SPACING OF CHECK DAMS BASED ON GRADE OF THE SWALE OR DITCH. TOP OF DOWNSTREAM DAM SHALL BE AT SAME ELEVATION AS TOE OF UPSTREAM DAM.

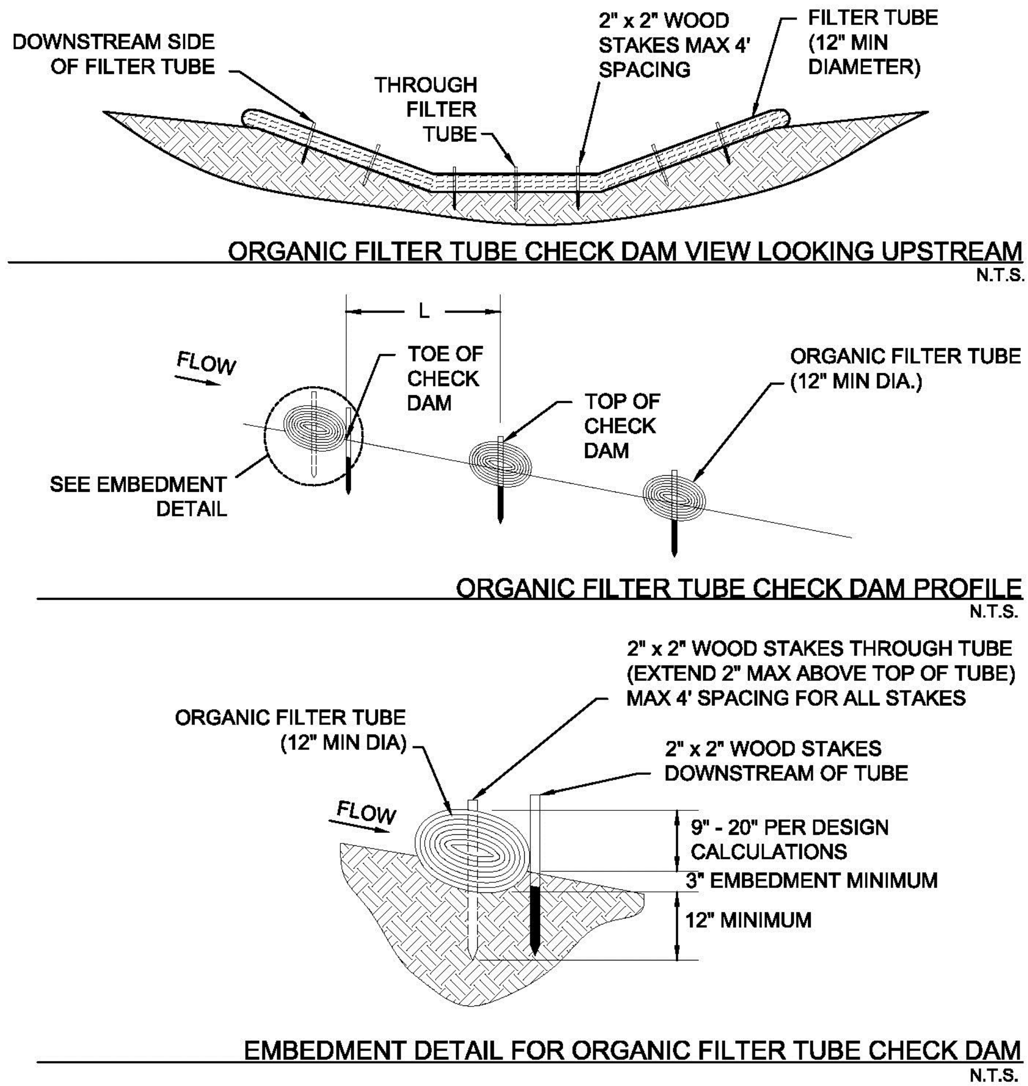
Figure 2.2 Schematics of Rock Bag Check Dams



NOTES: ACTUAL DIMENSIONS OF THE CHECK DAMS SHALL BE DESIGNED BASED ON FLOW CONDITIONS IN THE DRAINAGE SWALE OR DITCH. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE CHECK DAMS.

- HEIGHT OF CHECK DAMS BASED ON SWALE OR DITCH DIMENSIONS AND FLOW CONDITIONS.
- SPACING OF CHECK DAMS BASED ON GRADE OF THE SWALE OR DITCH. TOP OF DOWNSTREAM DAM SHALL BE AT SAME ELEVATION AS TOE OF UPSTREAM DAM.

Figure 2.3 Schematics of Sack Gabion Check Dams
(Source: Modified from Texas Department of Transportation Detail Sheet EC (2)-93)



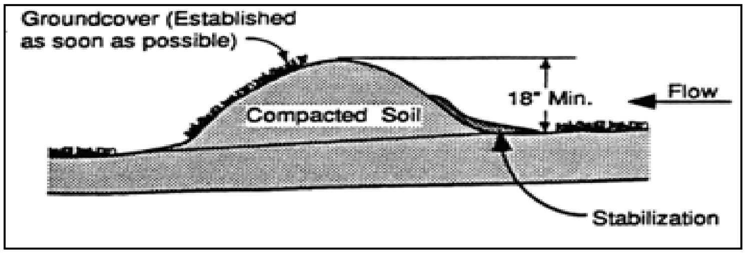
NOTES: ACTUAL DIMENSIONS OF THE CHECK DAMS SHALL BE DESIGNED BASED ON FLOW CONDITIONS IN THE DRAINAGE SWALE OR DITCH. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE CHECK DAMS.

- HEIGHT OF CHECK DAMS BASED ON SWALE OR DITCH DIMENSIONS AND FLOW CONDITIONS.
- SPACING OF CHECK DAMS BASED ON GRADE OF THE SWALE OR DITCH. TOP OF DOWNSTREAM DAM SHALL BE AT SAME ELEVATION AS TOE OF UPSTREAM DAM.

Figure 2.4 Schematics of Organic Filter Tube Check Dams
(Source: Modified from City of Plano BMP S-7)

2.2 Diversion Dike

Erosion Control



Description: A diversion dike is a compacted soil mound, which redirects runoff to a desired location. The dike is typically stabilized with natural grass for low velocities or with stone or erosion control mats for higher velocities.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum 1 foot flow depth at the dike for a 2-year return period design storm peak flow
- Side slopes 3:1 or flatter
- Minimum 2 feet top width
- Minimum embankment height of 18 inches measured from toe of slope on upgrade side
- Maximum contributing drainage area of 5 acres or less

ADVANTAGES / BENEFITS:

- Easy to install during early grading operations
- Very effective in reducing erosion at a reasonable cost
- Can be used in combination with an interceptor swale

DISADVANTAGES / LIMITATIONS:

- Must be stabilized immediately after placement or the dike will become a sediment source
- Can be a hindrance to construction equipment moving on the site

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove silt
- Repair erosion on the face of the dike
- Provide additional stabilization if erosion occurs

TARGETED POLLUTANTS

● Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.95

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
○ Training
● Suitability for Slopes > 5%

Other Considerations:

- *None*

2.2.1 Primary Use

The primary use of diversion dikes is to prevent erosion by diverting runoff away from steep slopes and disturbed areas. The diversion dike is normally used to intercept offsite flow upstream of the construction area and direct the flow around the disturbed soils. It can also be used downstream of the construction area to direct flow into a sediment control, such as a sediment basin or protected inlet. The diversion dike serves the same purpose as an interceptor swale and, based on the topography of the site, can be used in combination with an interceptor swale.

2.2.2 Applications

By intercepting runoff before it has the chance to cause erosion, diversion dikes are very effective in reducing erosion at a reasonable cost. They are applicable to a large variety of projects including site developments and linear projects, such as roadways and pipeline construction. Diversion dikes are normally used as upslope perimeter controls for construction sites with large amounts of offsite flow that needs to be re-directed around the construction site. They can also be used as a downslope perimeter control to direct runoff from the disturbed area to a sediment control.

Used in combination with swales, the diversion dike can be quickly installed with a minimum of equipment and cost, using the swale excavation material to construct the dike. No sediment removal technique is required if the dike is properly stabilized and the runoff is intercepted prior to crossing disturbed areas.

Significant savings in sediment controls can be realized by using diversion dikes to direct sheet flow from disturbed areas to a central sediment control, such as a sediment basin or other sediment trap, instead of installing a series of high-maintenance linear controls. Dikes can also be used to direct runoff from disturbed areas to a filtration device, passive treatment system, or active treatment system when these are necessary to attain required levels of sediment removal.

2.2.3 Design Criteria

- The maximum contributing drainage area should be 5 acres or less depending on site conditions.
- Maximum depth of flow at the dike shall be 1 foot based on a 2-year return period design storm peak flow.
- Side slopes of the diversion dike shall be 3:1 or flatter.
- Side slopes of the diversion dike may be 2:1 for dike installations to be used less than 3 months, if the dike is within an area protected by perimeter controls.
- Minimum width at the top of the dike shall be 2 feet.
- Minimum embankment height shall be 18 inches as measured from the toe of slope on the upgrade side of the berm.
- For grades less than 2 percent and velocities less than 6 feet per second, the minimum required channel stabilization shall be grass, erosion control blankets, or anchored mulch. For grades in excess of 2 percent or velocities exceeding 6 feet per second, stabilization is required in the form of turf reinforcement mats (or riprap with appropriate size, gradation, and thickness depending on flow conditions). Velocities greater than 8 feet per second will require approval by the local municipality and is discouraged.
- Refer to *Section 2.9 Vegetation* for design criteria and guidance on establishing vegetation in the swale.
- The dikes shall remain in place until all disturbed areas that are protected by the dike are permanently stabilized unless other controls are put into place to protect the disturbed area.
- The flow line at the dike shall have a positive grade to drain to a controlled outlet.

- Diverted runoff from a disturbed or exposed upland area shall be conveyed to a sediment-trapping device.
- The soil for the dike shall be placed in lifts of 8 inches or less and be compacted to 95 percent standard proctor density using ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort.
- Soil used in construction of the dike can be onsite material. It should be free of rocks larger than three inches in diameter and should be clay, silty clay or sandy clay with a plasticity index greater than 25. If only low PI material is available, it will be necessary to armor the slopes with stone or geotextile to prevent erosion of the dike.
- An interceptor swale may be installed on the upslope side of the diversion dike. Refer to *Section 2.4 Interceptor Swale* for swale design criteria.

2.2.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.7 Diversion Dike.

2.2.5 Inspection and Maintenance Requirements

Dikes should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to determine if silt is building up behind the dike or if erosion is occurring on the face of the dike. Silt shall be removed in a timely manner. If erosion is occurring on the face of the dike, the face of the slopes shall either be stabilized with mulch or seeding or the slopes shall be flattened.

2.2.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

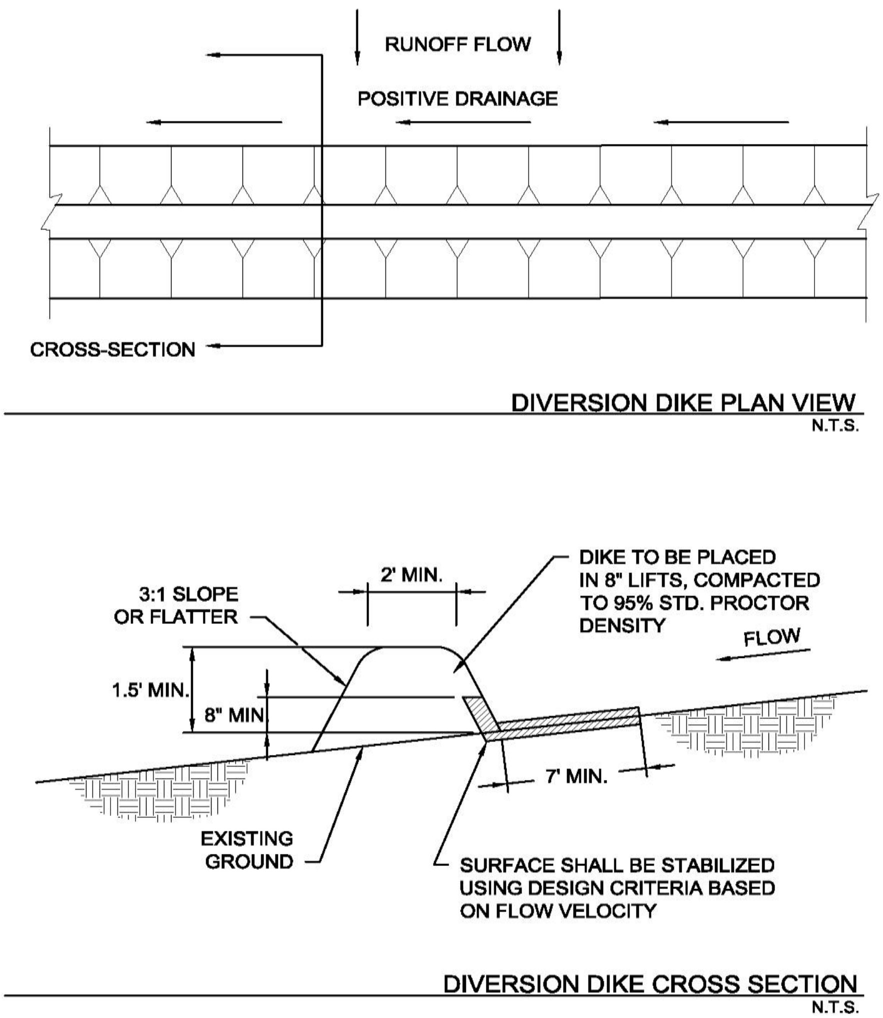


Figure 2.5 Schematics of Diversion Dike

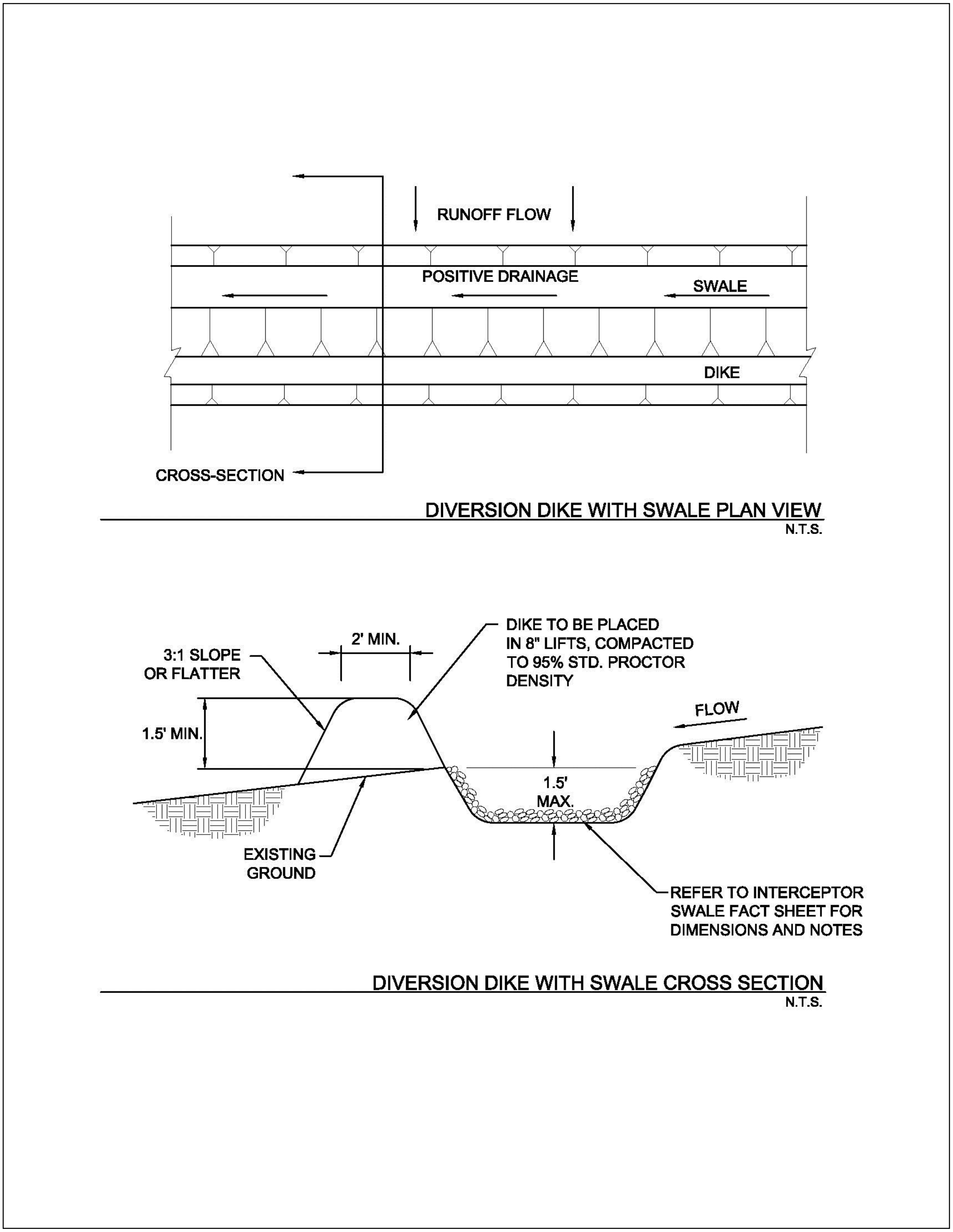
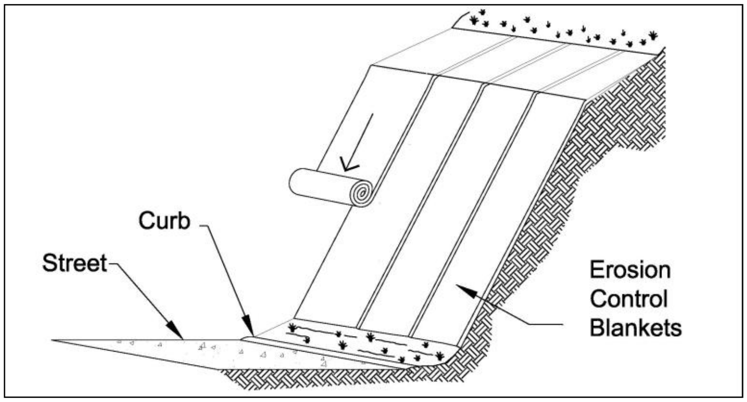


Figure 2.6 Schematics of Diversion Dike with Swale

2.3 Erosion Control Blankets

Erosion Control



Description: An erosion control blanket (ECB) is a temporary, degradable, rolled erosion control product that reduces soil erosion and assists in the establishment and growth of vegetation. ECBs, also known as soil retention blankets, are manufactured by many companies and are composed primarily of processed, natural, organic materials that are woven, glued, or structurally bound together with natural fiber netting or mesh on one or both sides.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- ECB selected based on slope, flow rate and length of service
- Specify preparation of soil surface to ensure uniform contact with blanket
- Installation and anchoring according to manufacturer's recommendations

ADVANTAGES / BENEFITS:

- Holds seed and soil in place until vegetation is established
- Effective for slopes, embankments and small channels

DISADVANTAGES / LIMITATIONS:

- Not for use on slopes greater than 2:1 or in channels with shear stresses greater than 2.0 pounds per square foot

MAINTENANCE REQUIREMENTS:

- Replace or re-anchor loosened blankets
- Remove sediment deposited on blankets

TARGETED POLLUTANTS

● Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Waste

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.90 (*Ground cover*)
Fe=0.65
(*Perimeter w/o vegetation*)

IMPLEMENTATION CONSIDERATIONS

● Capital Costs
◒ Maintenance
◒ Training
● Suitability for Slopes > 5%

Other Considerations:

- *Life expectancy, partial degradation, and mowing/ maintenance issues for ECBs left in place as part of final stabilization*

2.3.1 Primary Use

Erosion control blankets (ECBs) are used to hold seed and soil in place until vegetation is established on disturbed areas. They can be used on many types of disturbed areas, but are particularly effective for slopes and embankments and in small drainage swales.

ECBs seeded for vegetation may be used as a perimeter control. When used in combination with other sediment barriers, such as silt fence or organic filter tubes, blankets may be used as a perimeter control with or without vegetation.

2.3.2 Applications

ECBs may be used on many types of disturbed areas but are most applicable on gradual to steep (2:1) cut/fill slopes and in swales and channels with low to moderate flow velocities. In these applications they may provide temporary stabilization by themselves or may be used with seeding to provide final stabilization. ECBs are also used to establish vegetation in channels where velocities are less than 6.0 feet per second.

When seeded for establishment of vegetation, ECBs can be an effective perimeter along the down slope side of linear construction projects (roads and utilities). ECBs with vegetation are also used as perimeter controls for new development, particularly at the front on residential lots in new subdivisions. ECBs are an effective aid in establishing vegetated filter strips.

2.3.3 Design Criteria

- The designer shall specify the manufacturer, type of erosion control blanket to be used, and dimensioned limits of installation based on the site topography and drainage.
- The type and class of erosion control blanket must be specified in accordance with the manufacturer's guidance for the slope of the area to be protected, the flow rate (sheet flow on cut/fill slopes) or velocity (concentrated flow in swales) of stormwater runoff in contact in with the ECB, and the anticipated length of service.
- ECBs should meet the applicable "Minimum Performance Standards for TxDOT" as published by TxDOT in its "Erosion Control Report" and/or be listed on the most current annual "Approved Products List for TxDOT" applicable to TxDOT Item 169 Soil Retention Blanket and its Special Provisions.
- ECBs shall be installed vertically down slope (across contours) on cut/fill slopes and embankments and along the contours (parallel to flow) in swales and drainage ditches.
- ECBs designed to remain onsite as part of final stabilization shall have netting or mesh only on one side (the exposed side) of the ECB. The ECB shall be installed with the side that does not have netting or mesh in contact with the soil surface. All materials in the ECB, including anchors, should be 100 percent biodegradable within three years.
- On cut/fill slopes and drainage ditches or swales designed to receive erosion control blankets for temporary or final stabilization, installation of the ECBs shall be initiated immediately after completing grading of the slope or drainage way, and in no case later than 14 days after completion of grading these features. Do not delay installation of ECBs on these highly-erodible areas until completion of construction activities and stabilization of the remainder of the site.
- Unless the ECB is seeded to establish vegetation, perimeter control applications shall be limited to thirty foot wide drainage areas (i.e. linear construction projects) for an 8 foot width of ECB. When seeded for vegetation, use of ECBs for perimeter control shall follow the criteria in the *Section 3.15 Vegetated Filter Strips and Buffers*.
- Prior to the installation of the ECB, all rocks, dirt clods, stumps, roots, trash and any other obstructions that would prevent the ECB from lying in direct contact with the soil shall be removed.

- Anchor trenching shall be located along the top of slope of the installation area, except for small areas with less than 2 percent slope.
- Installation and anchoring shall conform to the recommendations shown within the manufacturer's published literature for the erosion control blanket. Anchors (staples) shall be a minimum of 6 inches in length and 1 inch wide. They shall be made of 11-gauge wire, or equivalent, unless the ECB is intended to remain in place with final stabilization and biodegrade.
- Particular attention must be paid to joints and overlapping material. Overlap along the sides and at the ends of ECBs should be per the manufacturer's recommendations for site conditions and the type of ECB being installed. At a minimum, the end of each roll of ECB shall overlap the next roll by 3 feet and the sides of rolls shall overlap 4 inches.
- After installation, the blankets should be checked for uniform contact with the soil, security of the lap joints, and flushness of the staples with the ground.
- When ECBs are installed to assist with establishing vegetation, seeding shall be completed before installation of the ECB. Criteria for seeding are provided in *Section 2.9 Vegetation*.
- Turf Reinforcement Mats should be used instead of ECBs for permanent erosion control and for stabilizing slopes greater than 2:1.
- ECBs are limited to use in swales and channels that have shear stresses of less than 2.0 pounds per square foot. Turf reinforcement mats shall be used in open channels with higher shear stresses.

2.3.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.15 Erosion Control Blankets and in Item 169 of the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT, 2004).

2.3.5 Inspection and Maintenance Requirements

Erosion control blankets should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for bare spots caused by weather or other events. Missing or loosened blankets must be replaced or re-anchored.

Check for excess sediment deposited from runoff. Remove sediment and/or replace blanket as necessary. In addition, determine the source of excess sediment and implement appropriate measures to control the erosion. Also check for rill erosion developing under the blankets. If found, repair the eroded area. Determine the source of water causing the erosion and add controls to prevent its reoccurrence.

2.3.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. The designer is responsible for working with ECB manufacturers to ensure the proper ECB is specified based on the site topography and drainage. Installation measures should be dictated by the ECB manufacturer and are dependent on the type of ECB installed. Manufacturer's recommendations for overlap, anchoring, and stapling shall always be followed. Criteria shown here are applicable only when they are more stringent than those provided by the manufacturer.

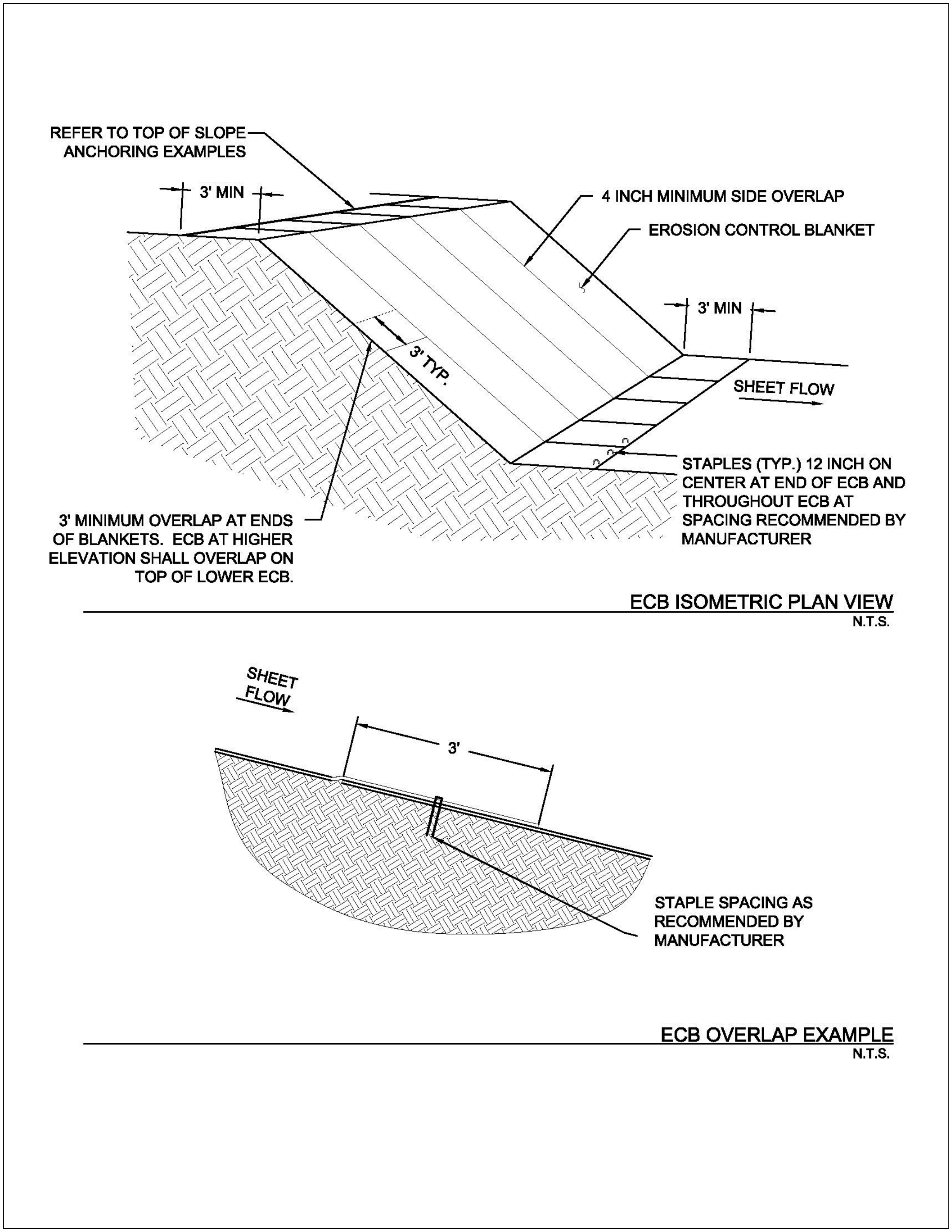
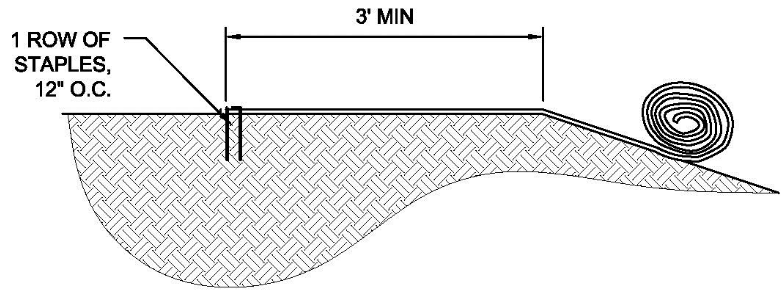
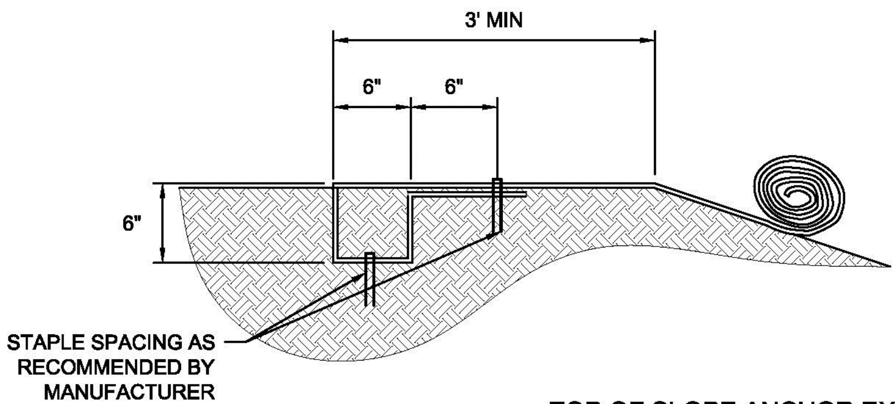


Figure 2.7 Schematics of Erosion Control Blankets



TOP OF SLOPE ANCHOR EXAMPLE 1
N.T.S.



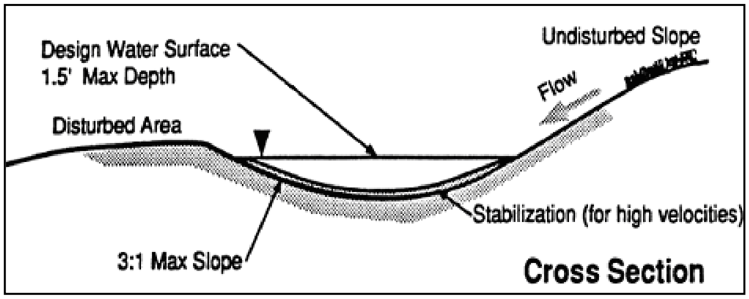
TOP OF SLOPE ANCHOR EXAMPLE 2
N.T.S.

NOTE: ANCHORING METHODS PROVIDED ARE EXAMPLES OF THE TYPE OF ANCHORING THE ECB MANUFACTURER MAY RECOMMEND. THERE ARE MORE THAN A DOZEN DIFFERENT TOP OF SLOPE ANCHORING METHODS BASED ON TYPE OF ECB, TYPE OF SOIL, SPECIFIED PERFORMANCE PERIOD, SLOPE STEEPNESS, ETC. ALWAYS FOLLOW THE MANUFACTURER'S RECOMMENDATIONS FOR ANCHORING BASED ON THE SITE-SPECIFIC APPLICATION.

Figure 2.8 Anchor Examples for Erosion Control Blankets
(Sources: American Excelsior Company and Western Excelsior Corporation)

2.4 Interceptor Swale

Erosion Control



Description: An interceptor swale is a small v-shaped, trapezoidal, or parabolic channel that collects runoff and directs it to a desired location. It can either have a natural grass lining or, depending on slope and design velocity, a protective lining of erosion control blankets, turf reinforcement mats, or rock riprap.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum flow depth of 1.5 feet for a 2-year, 24-hour design storm
- Side slopes 3:1 or flatter
- Minimum freeboard of 6 inches
- Maximum velocity of 8 fps with stabilization
- Maximum contributing drainage area of 5 acres

ADVANTAGES / BENEFITS:

- Prevents erosion and reduces cost of sediment controls by directing "clean" runoff around disturbed areas
- Easy to install during early grading operations

DISADVANTAGES / LIMITATIONS:

- Must be stabilized immediately after excavation or the swale will become a sediment source
- May be unsuitable to the site conditions (too flat or steep)

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove debris or other obstructions so as not to diminish flow capacity
- Repair damage from storms or normal construction activities such as tire ruts

TARGETED POLLUTANTS

● Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Waste

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.95

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
○ Training
● Suitability for Slopes > 5%

Other Considerations:

- *None*

2.4.1 Primary Use

The primary use of interceptor swales is to prevent erosion by diverting runoff around disturbed areas and steep slopes. The interceptor swale can either be used to direct sediment-laden flow from disturbed areas into a sediment control or to direct 'clean' runoff from upslope areas around the disturbed areas. Since the swale is easy to install during early grading operations, it can serve as the first line of defense in reducing sediment by reducing runoff across disturbed areas. An interceptor swale reduces the requirements for structural measures to capture sediment from runoff, since the volume of runoff is reduced. By intercepting sediment laden flow downstream of the disturbed area, runoff can be directed into a sediment basin or other control for sedimentation as opposed to long runs of silt fence or other filtration method.

2.4.2 Applications

Common applications for interceptor swales include roadway projects, site development projects with substantial offsite flow onto the construction site, and sites with a large area(s) of disturbance. The swale can be used in conjunction with diversion dikes to intercept flows. Temporary swales can be used throughout the project to direct flows away from staging, storage, and fueling areas to minimize the potential for construction materials and wastes to come into contact with runoff.

Runoff from disturbed areas that flows into a swale and flows within unstabilized (bare soil) swales must be routed into a sediment control such as a sediment basin. Dikes can also be used to direct runoff from disturbed areas to a filtration device, passive treatment system, or active treatment system when these are necessary to attain required levels of sediment removal.

Vegetated swales are an effective final stabilization technique if used to permanently direct flows around steep, easily eroded, slopes. The vegetation in the swale also effectively filters both sediment and other pollutants while reducing erosion potential.

2.4.3 Design Criteria

- Design calculations are required for the use of this control. The designer shall provide drainage computations, channel shape, channel dimensions, and channel slopes for each application.
- The maximum contributing drainage area should be 5 acres or less depending on site conditions.
- Maximum depth of flow in the swale shall be 1.5 feet based on a 2-year, 24-hour design storm. Positive overflow must be provided to accommodate larger storms.
- For permanent swales, the 1.5 feet maximum depth can be increased as long as provisions for public safety are implemented.
- The maximum contributing drainage area should be 5 acres or less depending on site conditions.
- Channels may be trapezoidal, parabolic, or v-shaped; however v-shaped channels may be difficult to stabilize, so they are generally used only where the volume and rate of flow is low.
- Side slopes of the swale shall be 3:1 or flatter.
- Side slopes of the interceptor swale may be 2:1 for swales to be used less than 3 months if flows in the swale are directed to a sediment control.
- Minimum design channel freeboard shall be 6 inches.
- For grades less than 2 percent and velocities less than 6 feet per second, the minimum required channel stabilization shall be grass, erosion control blankets or anchored mulch. For grades in excess of 2 percent or velocities exceeding 6 feet per second, stabilization is required in the form of turf reinforcement mats (or riprap with appropriate size, gradation, and thickness depending on flow conditions). Velocities greater than 8 feet per second will require approval by the local municipality and is discouraged.

- Refer to *Section 2.9 Vegetation* for design criteria and guidance on establishing vegetation in the swale.
- Check dams can be used to reduce velocities in steep swales. See *Section 2.1 Check Dam* for design criteria.
- Interceptor swales must be designed for flow capacity based on Manning's Equation to ensure a proper channel section. Alternate channel sections may be used when properly designed and accepted.
- Consideration must be given to the possible impact that any swale may have on upstream or downstream conditions.
- The outlet (discharge point) of the swale shall be designed to have non-erosive velocities or designed with velocity dissipation devices.
- Diverted runoff from a disturbed area or other construction activity shall be conveyed to a sediment-trapping device.
- A diversion dike may be used with an interceptor swale. Refer to *Section 2.2 Diversion Dike* for dike design criteria.

2.4.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.6 Interceptor Swale.

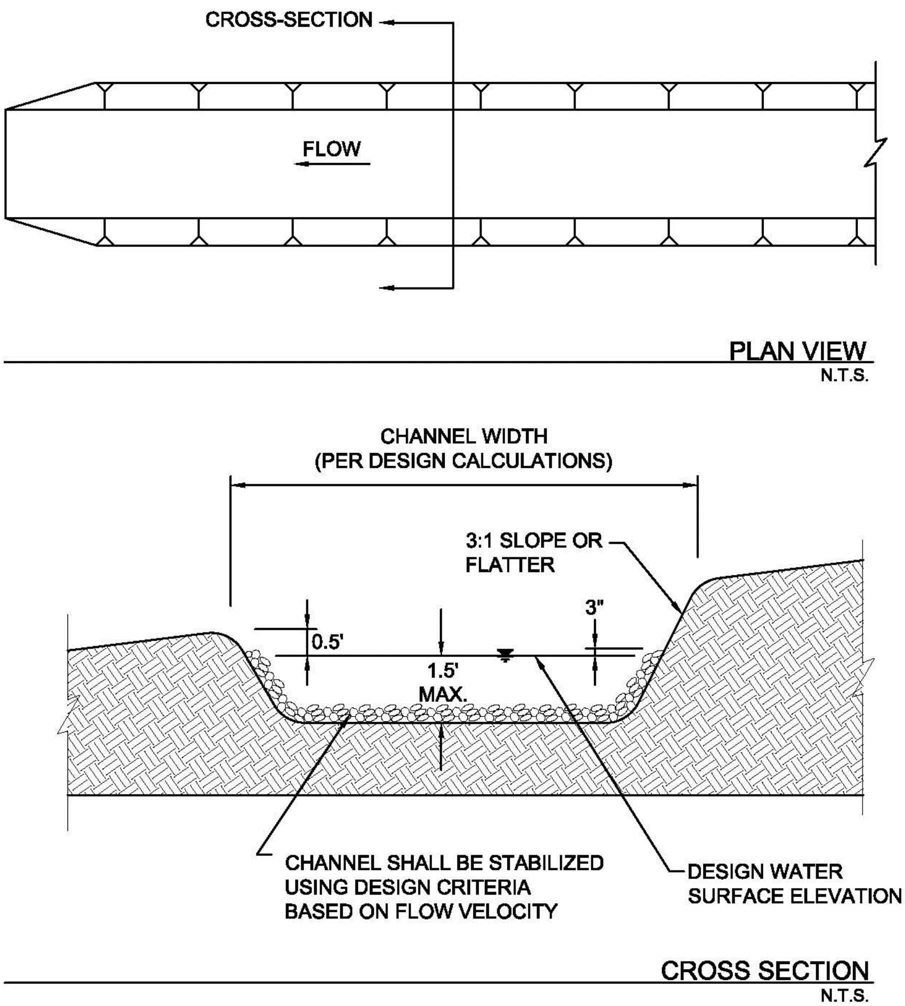
2.4.5 Inspection and Maintenance Requirements

Swales should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to locate and repair any damage to the channel or to clear debris or other obstructions so as not to diminish flow capacity. Damage from storms or normal construction activities such as tire ruts or disturbance of swale stabilization shall be repaired as soon as practical. Accumulated sediment deposited from water in the swale should be removed regularly to maintain the hydraulic capacity of the swale.

2.4.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



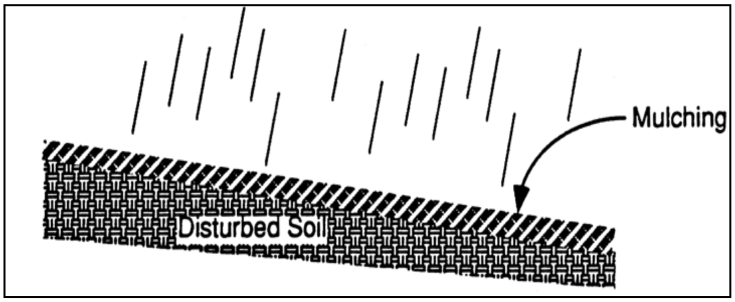
NOTE: DIMENSIONS OF THE SWALE SHALL BE DESIGNED BASED ON FLOW CONDITIONS. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE SWALE.

- SIZE OF CONTRIBUTING DRAINAGE AREA
- DESIGN STORM
- SWALE CROSS SECTION DIMENSIONS AND SIDE SLOPES
- GRADE OF FLOW LINE IN THE SWALE
- DESIGN VELOCITY IN SWALE

Figure 2.9 Schematics of Interceptor Swale

2.5 Mulching

Erosion Control



Description: Mulching is the application of a uniform layer of organic material over barren areas to reduce the effects of erosion from rainfall. Types of mulch include compost mixtures, straw, wood chips, bark, or other fibers. Commercialized surface treatments that combine straw or other mulch material with organic or inorganic soil binding systems are also available and are particularly useful on steep slopes.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Specify even, uniform application
- Thickness of 1 to 2 inches, depending on application
- Application criteria specific to type of mulch
- Anchor mulch on slopes of 3:1 to 1.5:1
- Do not use mulch on slopes steeper than 1.5:1

ADVANTAGES / BENEFITS:

- Provides immediate stabilization of bare areas
- May be used with seeding for final stabilization
- Decreases soil moisture loss
- Decreases velocity of sheet flow
- Reduces volume of sediment-laden flow

DISADVANTAGES / LIMITATIONS:

- Subject to removal by wind or water
- Results in lower soil temperature, which may yield longer seed germination periods
- Should not be applied within the ordinary high-water mark of natural surface waters or within the design flow depth of constructed ditches and channels

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Replace regularly in high traffic areas to maintain uniform thickness
- Maintain a stockpile of excess mulch at the site to repair problem spots

TARGETED POLLUTANTS

◒ Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.75-0.90
(Depends on coverage)

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
○ Training
◒ Suitability for Slopes > 5%

Other Considerations:

- *Availability of materials for mulch*
- *Application depends on slope*

2.5.1 Primary Use

Mulch may be used by itself to temporarily stabilize bare areas or with seed to establish final stabilization of bare areas. Mulch protects the soil from erosion and moisture loss by lessening the effects of wind, water, and sunlight. It also decreases the velocity of sheet flow, thereby reducing the volume of sediment-laden water flow leaving the mulched area.

2.5.2 Applications

Mulch may be applied on most areas disturbed by construction that require surface protection including:

- Freshly seeded or planted areas;
- Disturbed areas at risk of erosion due to the time period being unsuitable for growing vegetation;
- Disturbed areas that are not conducive to vegetation for temporary stabilization; or
- Steep slopes of 3:1 to 1.5:1, provided the mulch is anchored to the soil by use of soil stabilizers, netting, or crimping.

Mulch is frequently applied with seeding for vegetation. In these cases, refer to *Section 2.9 Vegetation* for related criteria that may affect mulching.

Mulch may also be applied with commercially available polymers for soil surface treatment to bind the mulch with the soil. This method is particularly useful on steep slopes. Related criteria are available in *Section 2.7 Soil Surface Treatments*.

2.5.3 Design Criteria

General

- Specific design information is required for the use of this control. The designer shall specify the type of mulch to be used, the application rate and/or thickness, and the type of anchoring (if applicable) based on site conditions.
- Choice of mulch depends largely on slope and soil type, in addition to availability of materials.
- Netting, adhesive polymers, or other methods of anchoring the mulch are required on slopes of 3:1 to 1.5:1. Do not use mulch on slopes steeper than 1.5:1.
- Mulch should be applied in an even and uniform manner where concentrated water flow is negligible. Do not apply mulch within the ordinary high-water mark of natural surface waters or within the design flow depth of constructed ditches and channels.
- Hay should not be used as mulch.
- Organic mulches may be distributed by hand or by mechanical means, provided a uniform thickness is achieved.
- When mulch is used with vegetation for final stabilization, fertilization and soil treatment for vegetation establishment should be done prior to placement of mulch, with the exception of hydroseeding or when seed is distributed following straw mulch spread during winter months.
- Table 2.1 on the following page contains a summary of mulch types and general guidelines.

Table 2.1 Mulch Standards and Guidelines			
Mulch Material	***Quality Standards***	***Application Rates***	***Remarks***
Straw	Air-dried, free of mold and not rotten. Certified Weed Free.	1.5 to 2 tons per acre	Cost-effective when applied with adequate thickness. Straw must be held in place by crimping, netting, or soil stabilizer.
Chipped Site Vegetation	Should include gradation from fine to coarse to promote interlocking properties. Must be free of waste materials such as plastic bags, metal debris, etc.	10 to 12 tons per acre	Cost-effective method to dispose of vegetative debris from site. Best application is for temporary stabilization where construction will resume. Use cautiously on areas where vegetation will be established, as wood chips will deplete soil nitrogen.
Erosion Control Compost (Wood Chip and Compost Mixture)	Shall meet the Physical Requirements in Table 1 of TxDOT Special Specification 1001.	Approx. 10 tons per acre	Special caution is advised regarding the source and composition of wood mulches. Ensure compost is free of herbicides. Ensure wood chips are from unpainted and untreated wood.
Hydraulic Mulch	Must not contain sawdust, cardboard, paper, paper byproducts, plastics, or synthetics. No petroleum-based tackifiers.	Follow the manufacturer's recommendations. Application rate increases with slope steepness.	May be particularly effective on slopes steeper than 3:1. Ensure wood fibers are from unpainted and untreated wood.

Straw Mulch

- Straw mulch shall be free of weed and grass seed.
- Straw mulch shall be air-dried, free of mold, and not rotten.
- Straw fibers shall be a minimum of 4 inches and a maximum of 8 inches in length.
- Straw mulch must be anchored by using a tractor-drawn crimper to punch into the soil, by placing degradable netting above the mulch, or by application of a soil stabilizer (*Section 2.7 Soil Surface Treatments*).

Chipped Site Vegetation

- Chipped site vegetation is suitable mulch for temporary stabilization before construction will resume in an area of the construction site.
- Ensure the cleared vegetation is free of trash, litter, and debris prior to chipping.

- Chipped pieces shall be a minimum of 2 inches and a maximum of 6 inches in length.
- Chipped woody vegetation that is greater than 50% wood chips by volume may result in mulch that depletes nitrogen in the soil. It is useful as mulch for temporary stabilization where construction activity will resume and result in removal of the mulch. However, it should be used with care on areas where vegetation will be established for final stabilization.
- Chipped vegetation that is greater than 50 percent wood chips by volume may require treatment with a nitrogen fertilizer when used for mulch with seeding.
- Chipped vegetation that includes green matter will include seeds. It should not be used on areas that have specific landscaping requirements.

Erosion Control Compost (Wood Chip and Compost Mixture)

- Wood chip and compost mixture used for mulch shall meet the criteria for Erosion Control Compost in TxDOT Special Specification 1001.
- Wood chips for the mixture shall be less than or equal to 5 inches in length with 95 percent passing a 2 inch screen and less than 30 percent passing a 1 inch screen. Mulch should not contain chipped manufactured boards or chemically treated wood such as particleboard, railroad ties, or similar treated wood.
- Compost for the mixture shall meet the Physical Requirements specified in Table 1 of 2004 TxDOT Special Specification 1001, Compost. It must be free of herbicides and other chemicals.
- Mixing of the Erosion Control Compost into the soil surface is allowed when vegetation is established for final stabilization, except for drill seeding, in which case it is best to leave the mulch as an undisturbed top layer.

Hydraulic Mulch (Including Bonded Fiber Matrix)

- Hydraulic mulch shall consist of a mixture of shredded wood fiber and a stabilizing binder. The mulch must not contain sawdust, cardboard, paper or paper byproducts.
- Shredded wood fiber shall be long strand, whole wood fibers that are:
  - Minimum of 25 percent of fibers 3/8 inch long;
  - Minimum of 50 percent held on a No. 25 sieve;
  - Free from paint, printing ink, varnish, petroleum products, seed germination inhibitors; and
  - Free from synthetic or plastic materials.
- Mulch binders may be organic or inorganic polymers. Asphaltic emulsions and other petroleum-based tackifiers shall not be used.
- The stabilizing emulsion must be nonflammable, non-toxic to aquatic organisms, and free from growth or germination inhibiting factors.
- Areas hydraulically mulched shall be protected from all traffic, including foot traffic, a minimum of 24 hours to allow the mulch to dry and cure. Depending on the mulch, up to 48 hours of protection may be required. Always follow manufacturer's recommendations.
- Hydraulic mulch provides limited to no protection until cured. Do not apply when rain is forecast within the next 24 hours.
- Hydraulic mulch may be particularly effective on slopes steeper than 3:1.

2.5.4 Design Guidance and Specifications

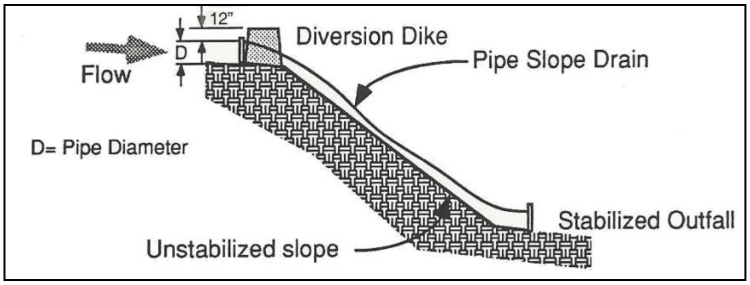
Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.16 Mulching. Specifications for compost may be found in Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges (TxDOT 2004) Item 161.

2.5.5 Inspection and Maintenance Requirements

Mulched areas should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for thin or bare spots caused by natural decomposition or weather related events. Mulch in high traffic areas should be replaced on a regular basis to maintain uniform protection. Excess mulch should be brought to the site and stockpiled for use during the maintenance period to dress problem spots.

2.6 Pipe Slope Drain

Erosion Control



Description: A pipe slope drain is a temporary or permanent pipeline, typically utilizing flexible pipe that conveys runoff down steep or unstabilized slopes without causing erosion. The drain is anchored on the upstream end with some form of headwall to limit erosion and secure the pipe.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum entrance grade of 3 percent
- Anchor upstream end with a headwall or similar device
- Secure pipe with hold down anchors spaced 10 feet on center
- Stabilize outlet and provide velocity dissipation so that released flow has a velocity less than 3 feet per second

ADVANTAGES / BENEFITS:

- Protects slopes from erosion caused by overland flow
- A series of pipes may be used to control drainage areas greater than 5 acres in size

DISADVANTAGES / LIMITATIONS:

- Drain can easily be damaged by construction traffic
- Difficult to secure pipe to the slope
- Can become clogged during large rain events causing water to overflow and create a serious erosion condition

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Repair damage to pipe joints
- Unclog pipe

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ○ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control

$\boxed{\textbf{Slope Protection}}$

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.95

IMPLEMENTATION CONSIDERATIONS

- ● Capital Costs
- ● Maintenance
- ○ Training
- ● Suitability for Slopes > 5%

Other Considerations:

- *Normally used in combination with interceptor swales or diversion dikes to direct flow*
- *Additional measures needed to remove sediment from runoff*

2.6.1 Primary Use

Pipe slope drains are used to protect graded slopes during establishment of temporary and final vegetation. They are used on sites with a long, unstabilized, steep slope area that is subject to erosion from overland flow. Drains are normally used in combination with interceptor swales or diversion dikes to direct the flow into the pipe. The pipe slope drain can provide service for a relatively large area. It does not treat the runoff; therefore if the runoff contains sediment from a disturbed area, treatment through a sediment control is required before the flow is released offsite.

2.6.2 Applications

Sites with large berms or grade changes, such as roadway embankments, are candidates for a pipe slope drain. Since provisions must be made to direct the flow into the pipe drain, some grading is normally required upstream of the pipe slope drain. Installed properly, slope erosion can be greatly reduced (but not entirely eliminated) through the use of the drain.

Pipe slope drains also require a stabilized outlet. This is critical since the velocities at the outfall are normally high. Velocity dissipators such as stone or concrete riprap are typically required to reduce the velocity and spread the flow, reducing erosion.

2.6.3 Design Criteria

- Design calculations and information are required for the use of this control. The designer shall provide drainage computations, pipe material, pipe size, and stone apron size for each application.
- The entrance to the pipe slope drain may be a standard corrugated, metal pre-fabricated, flared end section with an integral toe plate extending a minimum of 6 inches from the bottom of the end section.
- The grade of the entrance shall be 3 percent maximum.
- The diversion dike at the entrance shall have a minimum height of the pipe diameter plus 12 inches and a minimum width of 3 times the pipe diameter. Additional criteria are in *Section 2.2 Diversion Dike*.
- The drain pipe shall be made of any material, rigid or flexible, capable of conveying runoff. Regardless of material, the drain pipe shall be completely water-tight so that no water leaks onto the slope being protected.
- All sections of the pipe slope drain shall be connected using watertight collars or gasketed watertight fittings.
- If the upslope drainage area contributing flow to the pipe drain is disturbed or the collection swale/dike for the drain is not stabilized, flow from a pipe slope drain must be routed to a sediment control to remove suspended soil collected in these areas before being discharged from the site.
- The pipe shall be secured with hold down anchors spaced 10 feet on center.
- Temporary pipe slope drains are to be sized to accommodate runoff flows equivalent to a 10-year storm as calculated using the Rational Method and Manning's equation, but in no case shall pipes be sized smaller than shown on the following table.

Table 2.2 Pipe Slope Drain Minimum Diameters	
Minimum Pipe Size	*Maximum Contributing Drainage Area*
12 inches	0.5 Acres
18 inches	1.5 Acres
21 inches	2.5 Acres
24 inches	3.5 Acres
30 inches	5.0 Acres

- Maximum drainage areas for individual pipe slope drains shall be 5 acres. For areas larger than 5 acres, additional drains shall be added.
- Both the entrance and outfall of the pipe slope drain should be properly stabilized. Grass can normally be used at the entrance, but armor type stabilization such as stone or concrete riprap is normally required to address the high velocities of the outfall.
- A riprap lined apron shall be excavated to accept the discharge from the pipe and dissipate the energy of the flow. The width of the bottom of the apron shall be 3 times the pipe diameter, and the length shall be a minimum of 6 times the pipe diameter of the drain pipe.
- The riprap apron shall be a minimum of 12 inches in depth and shall be lined with well graded stone weighing between 50 and 150 pounds per stone at a minimum thickness of 12 inches. The top of the riprap apron shall be relatively flat (no slope) and flush with the surrounding ground.
- The apron shall be designed so that the released flow has a velocity less than 3 feet per second.

2.6.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.14 Pipe Slope Drain and in the Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges (TxDOT 2004) Item 506.2.B and 506.4.C.2.

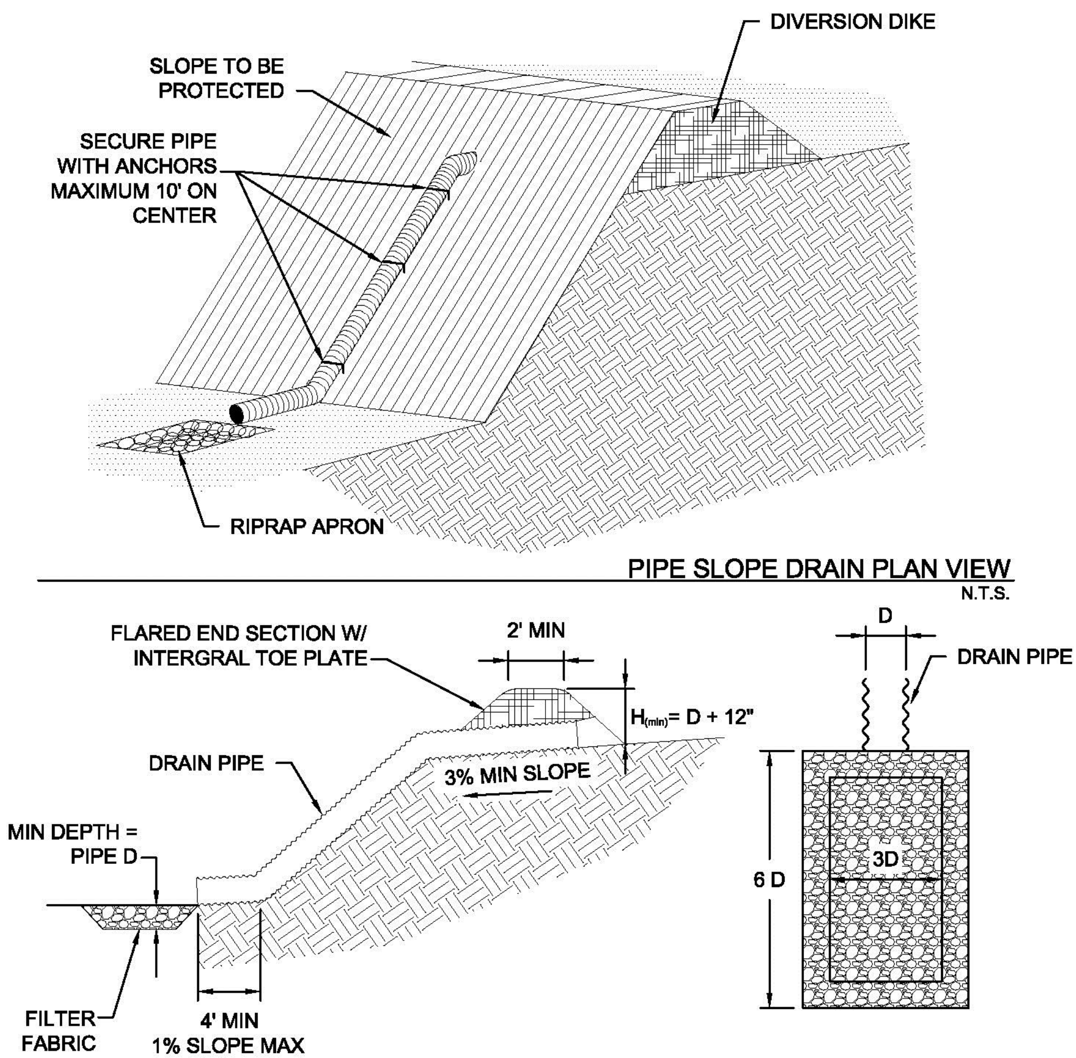
2.6.5 Inspection and Maintenance Requirements

Pipe slope drains should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to locate and repair any damage to joints or clogging of the pipe. In cases where the diversion dike has deteriorated around the entrance of the pipe, it may be necessary to reinforce the dike with sandbags or to install a concrete collar to prevent failure. Signs of erosion around the pipe drain should be addressed in a timely manner by stabilizing the area with erosion control blanket, turf reinforcement mats, riprap, concrete, or other acceptable methods.

2.6.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



RIPRAP APRON PLAN VIEW

RIPRAP SHALL CONSIST OF 50 TO 150 POUND STONES PLACED IN A LAYER OF NOT LESS THAN 12 INCHES. THE DEPTH OF THE APRON SHALL EQUAL THE PIPE DIAMETER BUT IN NO CASE SHALL IT BE LESS THAN 12 INCHES.

PIPE SLOPE DRAIN CROSS-SECTION AND APRON PLAN VIEW

N.T.S.

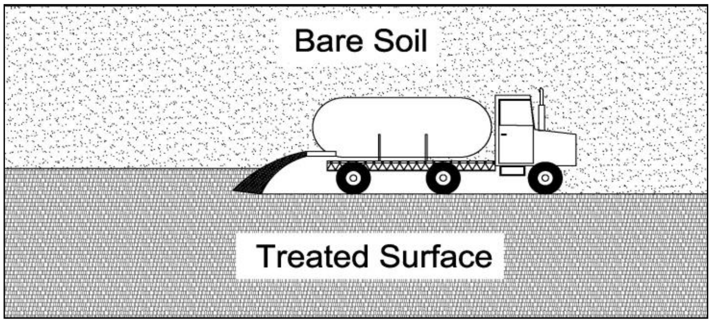
NOTE: DIMENSIONS OF THE PIPE SLOPE DRAIN AND APPURTENANCES SHALL BE DESIGNED BASED ON SITE TOPOGRAPHY AND FLOW CONDITIONS. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE PIPE SLOPE DRAIN.

- SIZE OF CONTRIBUTING DRAINAGE AREA
- DESIGN STORM
- PIPE MATERIAL AND SIZE
- DISCHARGE VELOCITY
- STONE SIZE AND DIMENSIONS OF RIPRAP APRON

Figure 2.10 Schematics of Pipe Slope Drain

2.7 Soil Surface Treatments

Erosion Control



Description: Soil surface treatments are measures applied to a bare soil surface to temporarily decrease the amount of soil lost to wind and water erosion. Substances typically applied to the soil surface are water and organic and inorganic palliatives. Soil surface treatments are also effective for the surfaces of temporary berms and stockpiles.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maintain the original ground cover as long as practical
- Select treatment method based on soil type, site conditions, and required duration of effectiveness
- Control traffic on areas being treated
- Apply water before start of work and repeat reguarly
- Select, dilute and apply palliatives according to manufacturer's recommendations

ADVANTAGES / BENEFITS:

- Prevents onsite and off-site impacts of dust deposition on roadways, drainage ways, or surface waters

DISADVANTAGES / LIMITATIONS:

- Sediment controls are still needed with soil surface treatments
- Effectiveness is temporary
- Control methods often require repeated applications
- Water has limited effectiveness on soils in wind erodibility groups 1 – 4 and 4L

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Reapply water and palliatives as needed

TARGETED POLLUTANTS

◒ Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
$\boxed{\textbf{Slope Protection}}$
Sediment Barrier
Channel Protection
$\boxed{\textbf{Temporary Stabilization}}$
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.10-0.90
(Depends on type of treatment)

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
◒ Training
◒ Suitability for Slopes > 5%

Other Considerations:

- *Worker protection for mixing, dilution, and application of some palliatives*

2.7.1 Primary Use

Surface treatments are used to reduce wind and water erosion by providing temporary stabilization of bare soil. They are primarily used where stabilization is needed for less than 12 months.

2.7.2 Applications

Soil surface treatments are applicable to any construction site where dust is created and there is the potential for air and water pollution from dust being blown off the site. The treatments are applicable to bare areas of soil, temporary soil berms, stockpiles, earth-moving activities, and demolition activities, all of which can be sources of dust.

The National Resources Conservation Service (NRCS) assigns a wind erodibility group to soils as shown in Table 2.3.

Table 2.3 NRCS Wind Erodibility Groups

Group	*Soil Type*	*Erosion Potential*
1	Sands, coarse sands, fine sands and very fine sands	Extremely erodible
2	Loamy sands, loamy fine sands, and loamy very fine sands	Very highly erodible
3	Sandy loams, coarse sandy loams, fine sandy loams, and very fine sandy loams	Highly erodible
4L	Calcareous loamy soils that are less than 35 percent clay and more than 5 percent finely divided calcium carbonate	Erodible
4	Clay, silty clays, clay loams and silty clay loams that are more than 35 percent clay	Moderately erodible
5	Loam soils that are less than 18 percent clay and less than 5 percent finely divided calcium carbonate and sandy clay loams and sandy clays that are less than 5 percent finely divided calcium carbonate	Slightly erodible
6	Loamy soils that are 18 to 35 percent clay and less than 5 percent finely divided calcium carbonate, except silty clay loams	Very slightly erodible
7	Siltly clay loams that are less than 35 percent clay and less than 5 percent finely divided calcium carbonate	Slightly erodible
8	Stony or gravelly soils	Not subject to wind erosion

Soil surface treatments for dust control will be most applicable to soils in groups 1 through 4 and 4L. If the soil type is unknown, the native soil type(s) at a site can be identified on the NRCS Web Soil Survey at: http://websoilsurvey.nrcs.gov/app/. The website also provides the wind erodibility group for native soils.

Consider the depth of grading activities when determining the applicable surface treatments. Soil type varies with depth. The surface soil may have low potential for wind erosion, but the soil at a lower depth may be highly erodible when it is exposed by grading operations.

2.7.3 Design Criteria

General

- The first design criterion for soil surface treatments is to minimize the area of disturbed soil that requires treatment.

- Limit clearing and grading to the areas of the site required for the immediate phase of construction. For larger sites, plan the work to be phased such that the total disturbed area is less than 10 acres at all times. If possible, design the site layout and grading to allow for street and utility construction without having to grade the entire site to balance cut and fill.
- Selection of the surface treatment should consider the length of time for which stabilization is needed.
- Natural (e.g. trees) windbreaks or artificial wind screens can be designed into the site to decrease wind erosion potential. Wind screens should be 3 to 5 feet in height. Porosity of the wind screens should be a minimum of 20 percent. Optimum performance is in the 40 percent to 60 percent porosity range.
- Wind screens should never be impermeable. The purpose of the screen is to disrupt the wind, not block it.
- Wind screens placed around stockpiles shall enclose three sides of the stockpile.

Water Treatments

- Water treatment shall be used only for decreasing wind erosion. It provides no protection from erosion due to stormwater runoff.
- Water treatment is appropriate for areas that are worked daily or at least as frequently as every week or two. Areas where construction activities will not occur for more than 14 days shall receive another type of surface treatment, such as a palliative, vegetation, or other treatment that provides temporary stabilization and protection from water erosion.
- Water shall be applied 15 to 20 minutes before start of work and re-applied throughout the day as necessary to prevent visible emissions.
- At a minimum, sprinkle bare areas with an amount of water and at a rate that will moisten the top two inches of soil without creating runoff.
- When grading activities are occurring during prolonged dry and windy periods, sufficient water should be applied to moisten soil to the depth of cut or equipment penetration. This may require installing portable piping and sprinklers in advance of grading.
- If construction activities include installing an irrigation system, install it in early phases of construction, where feasible, to use for dust control.
- Water treatments provide limited stabilization against wind erosion and no stabilization against water erosion. Sediment controls are required with water treatments.

Palliative Treatments

- Palliatives consist of liquids that react with soil particles and bonds them into a cohesive crust that provides temporary resistance to wind and water erosion. Palliative treatments are also called soil binders.
- The major groups of palliatives used for erosion control are polyacrylamide (PAM), guar-based (organic) compounds, and polyvinyl acetates (inorganic polymers). Numerous variations and mixes of these palliatives are available, each with its unique properties.
- Palliative treatments are appropriate for areas that require temporary stabilization for 3 to 12 months. Palliative treatments are highly effective in controlling wind erosion and moderately effective in controlling water erosion. Perimeter controls for sediment should remain in place until final stabilization.
- In general, areas stabilized with palliatives must be protected from traffic to be effective. Palliative treatments that can withstand traffic (pedestrian or vehicle) are available; however, they are more expensive. The designer should determine whether the site can be controlled to prevent traffic on the stabilized areas. This analysis should consider non-construction related traffic. Often, the public driving ATV's and bicycles on the site when construction is not active is the cause of stabilization

failure. In many cases, temporary chain-link fencing is less expensive than a palliative that can withstand traffic or re-applying a palliative to areas that have been disturbed.

- Selection of the palliative mix, dilution rate, and application rate should be based on the soil type, site conditions, climate, anticipated traffic on the treated area, and required duration of the stabilization.
- The designer should work with the supplier to develop a mix specific for the soil, climate, and site conditions. A successful application is highly dependent on the right proportions in the mix. An "off the shelf" mix should not be used.
- Palliatives are dependent on soil penetration to be effective. Compaction of soil prior to stabilization should be minimized. If compaction has occurred or the soil has high clay content, loosening of the surface may be necessary before applying the palliative.
- Do not apply palliatives in rainy conditions or when the soil has high moisture content. Verify that there is not rain in the forecast for the length of time recommended by the manufacturer to cure the palliative. Typically, a minimum of 24 hours is required.
- If the soil is excessively dry, pre-wetting may be necessary to ensure the palliatives do not cure too quickly.
- Palliative mixes may be supplied as a powder or a concentrated liquid. The designer should work with the supplier to establish exact dilution and application rates for the site. An application without enough water for the site and climate conditions will dry too quickly, and the soil particles will not bond properly. A too wet mix will result in a weaker bond that may not be sustained for the required duration of the stabilization.
- Palliatives should not be diluted until it is time for the palliative to be applied.
- Palliatives may be applied with mulch to stabilize slopes of 3:1 to 1.5:1. Additional criteria are in *Section 2.5 Mulching*.
- Palliatives may be mixed and applied with seed to establish vegetation. The palliative mix used for this application must be specified as one that is air and water permeable. The palliative will provide temporary stabilization until vegetation is established for final stabilization.

Vegetation Treatments

- If an area will not be disturbed by construction activities for a year or longer, vegetation is frequently the most cost-effective treatment.
- *Section 2.9 Vegetation* contains criteria for temporary stabilization with vegetation.

Other Treatments

- Gravel, recycled concrete or asphalt, or similar rock should be applied to temporary roads, contractor staging areas, employee parking lots and other portions of the site that receive daily traffic. The treatment will prevent dust and decrease the need for sediment controls on these areas during the duration of the construction project.
- Soil roughening, by driving tracked vehicles up and down slopes and across bare areas in irregular patterns, can be used to disrupt wind and water flow across the soil surface and decrease erosion for short periods of time. The track marks should be perpendicular to the predominate direction of water flow or wind.
- Similar to soil roughening, deep tillage (6 to 12 inches) in large open areas can significantly disrupt wind and drainage patterns to reduce erosion.
- Do not use "soil tackifiers" that are petroleum-based.

2.7.4 Design Guidance and Specifications

No specification for soil surface treatments is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

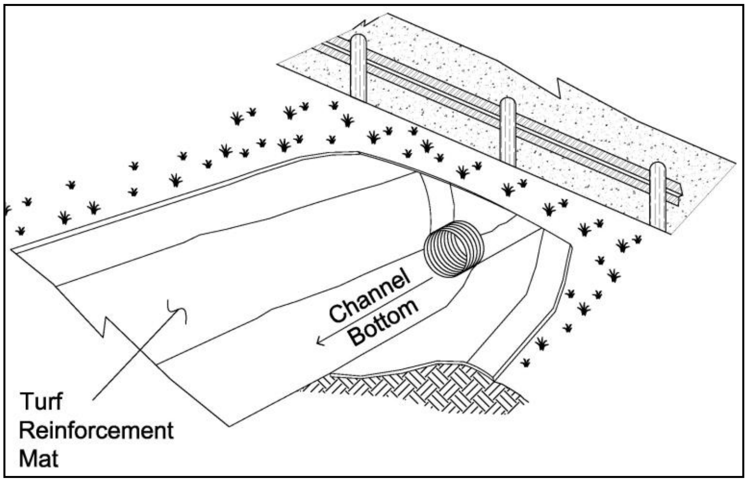
2.7.5 Inspection and Maintenance Requirements

Soil surface treatments should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Adequacy of watering for dust control should be visually monitored. If dust is observed, additional applications or different controls are needed.

Areas that have received a palliative treatment should be checked for breaks or eroded spots in the surface crust. This spots and areas that have been driven on or otherwise disturbed should be re-treated. Palliative treatments are intended to control sheet erosion. If rill erosion is detected during inspections, additional controls are needed.

2.8 Turf Reinforcement Mats

Erosion Control



Description: A turf reinforcement mat (TRM) is a long-term, non-degradable, rolled erosion control product that reduces soil erosion and assists in the establishment and growth of vegetation. TRMs, also known as flexible channel liners, are manufactured by many companies They are composed primarily of UV stabilized, geosythetic or geocomposite materials, netting and/or wire mesh, processed into a three dimensional reinforcement matrix. TRMs are designed to be permanent and for use in critical hydraulic conditions.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Select TRM based on flow velocities and shear stresses in the channel
- Installation and anchoring according to manufacturer's recommendations

ADVANTAGES / BENEFITS:

- Provides long-term stabilization of channels with high velocities and shear stresses
- Retains soil in a 3-dimensional matrix that facilitates establishment of vegetation

DISADVANTAGES / LIMITATIONS:

- Expensive
- Effectiveness is highly dependent on proper installation

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Replace or re-anchor loosened mats

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.90

TARGETED POLLUTANTS

● Sediment

○ Nutrients & Toxic Materials

○ Oil & Grease

○ Floatable Materials

○ Other Construction Wastes

IMPLEMENTATION CONSIDERATIONS

● Capital Costs

◒ Maintenance

◒ Training

● Suitability for Slopes > 5%

Other Considerations:

- *Long term maintenance*

2.8.1 Primary Use

Turf reinforcement mats (TRMs) are primarily used to provide temporary and final stabilization of channels where design discharges exert velocities and shear stresses that exceed the limits of mature vegetation. They are also used to stabilize steep slopes where it's difficult to establish vegetation.

2.8.2 Applications

TRMs provide long-term erosion protection in channels where flow conditions exceed the ability of vegetation alone to withstand erosive forces (grades in excess of 2 percent or velocities exceeding 6 feet per second). Turf reinforcement mats may provide channel protection for conditions of up to approximately 8 lbs/ft^2 sheer stress.

TRMs may also be used for short lengths of steep cut/fill slopes on which establishment of vegetation is difficult. TRMs also contain void spaces that can retain soil that would erode without protection, and thus give vegetation a change to establish.

2.8.3 Design Criteria

- The designer shall specify the manufacturer, type of TRM to be used, and dimensioned limits of installation based on the site topography and drainage.
- The type and class of TRM must be specified in accordance with the manufacturer's guidance for the slope of the area to be protected, the flow rate (sheet flow on cut/fill slopes) or velocity (concentrated flow in swales) of stormwater runoff in contact in with the TRM, shear stress, and the design life (duration) of the TRM.
- TRMs specified on projects should meet the applicable "Minimum Performance Standards for TxDOT" as published by TxDOT in its "Erosion Control Report." Alternatively, the TRM may be listed on the most current annual "Approved Products List for TxDOT" applicable to TxDOT Item 169 Soil Retention Mat and its Special Provisions.
- TRMs shall meet the following criteria when applied on slopes of 0.5:1 or flatter.
    - Minimum thickness of 0.25 inches using ASTM D6525 Standard Test Method for Measuring Nominal Thickness of Permanent Rolled Erosion Control Products.
    - UV stability of 80 percent at 500 hours using ASTM D4355 Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus.
    - Minimum tensile strength of 175 lbs/ft using ASTM D6818 Standard Test Method for Ultimate Tensile Properties of Turf Reinforcement Mats.
- TRMs shall be installed vertically down slope (across contours) on steep cut/fill slopes and embankments. In channels, TRMs shall be installed along the contours (parallel to flow) below the water surface elevation of the flood mitigation storm (100-year, 24-hour) and vertically across any steep slopes for high banks above the water surface elevation.
- On cut/fill slopes and channels designed to receive turf reinforcement mats for temporary or final stabilization, the installation of the TRMs shall be initiated immediately after completing grading of the slope or channel, and in no case later than 14 days after completion of grading these features. Do not delay installation of TRMs on these highly-erodible areas until completion of construction activities and stabilization of the remainder of the site.
- Prior to the installation of the TRM, all rocks, dirt clods, stumps, roots, trash and any other obstructions that would prevent the TRM from lying in direct contact with the soil shall be removed.
- Installation and anchoring shall conform to the recommendations shown within the manufacturer's published literature for the turf reinforcement mat. Anchors (staples) shall be a minimum of 6 inches in length and 1 inch wide. They shall be made of 8-gauge wire, or equivalent.

- The end of each TRM roll shall overlap the next end of the next roll by a minimum of 3 feet. Sides of rolls typically overlap a minimum of 4 inches.
- The perimeter of the TRM installation shall be anchored into a trench that is a minimum of 6 inches deep.
- The upstream end of TRMs used for channel protection shall be anchored a minimum of 12 inches, while the downstream end should be anchored 6 inches.
- Trenches shall be excavated for anchoring, followed by placement and tamping of fill on top of the mat.

2.8.4 Design Guidance and Specifications

Specifications for this item may be found in Item 169 of the Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges (TXDOT 2004).

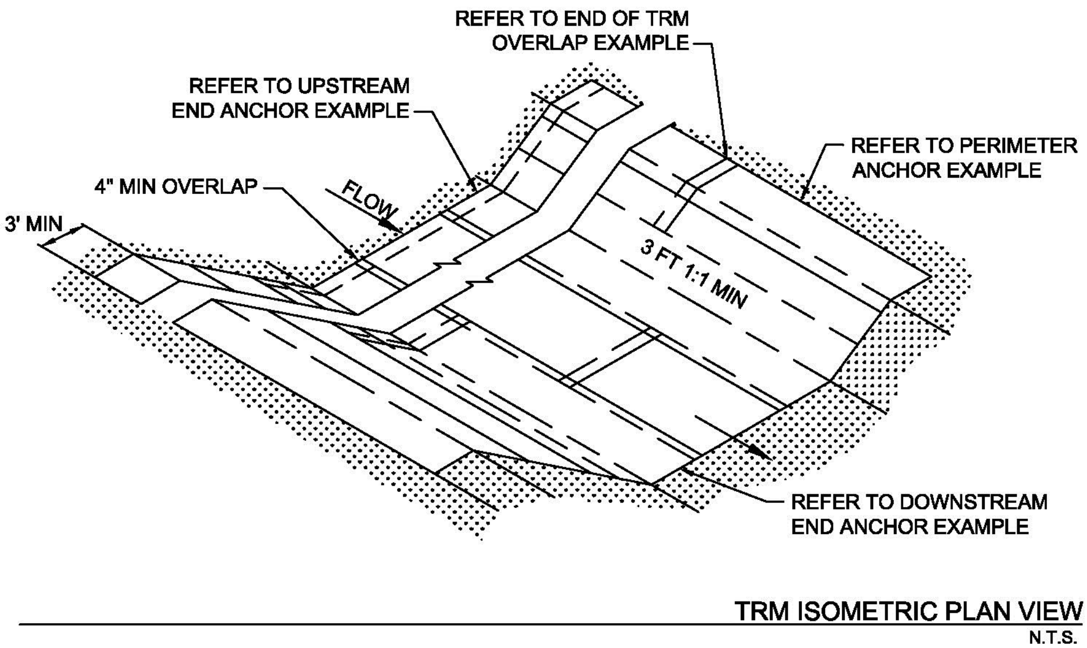
2.8.5 Inspection and Maintenance Requirements

Turf reinforcement mats should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for bare spots caused by weather or other events. The mats should be checked for uniform contact with the soil, security of the lap joints, and flushness of the staples with the ground. Missing or loosened mats must be replaced or re-anchored.

2.8.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. The designer is responsible for working with TRM manufacturers to ensure the proper TRM is specified based on the site topography and drainage. Installation measures should be dictated by the TRM manufacturer and are dependent on the type of TRM installed. Manufacturer's recommendations for overlap, anchoring, and stapling shall always be followed. Criteria shown here are applicable only when they are more stringent than those provided by the manufacturer.



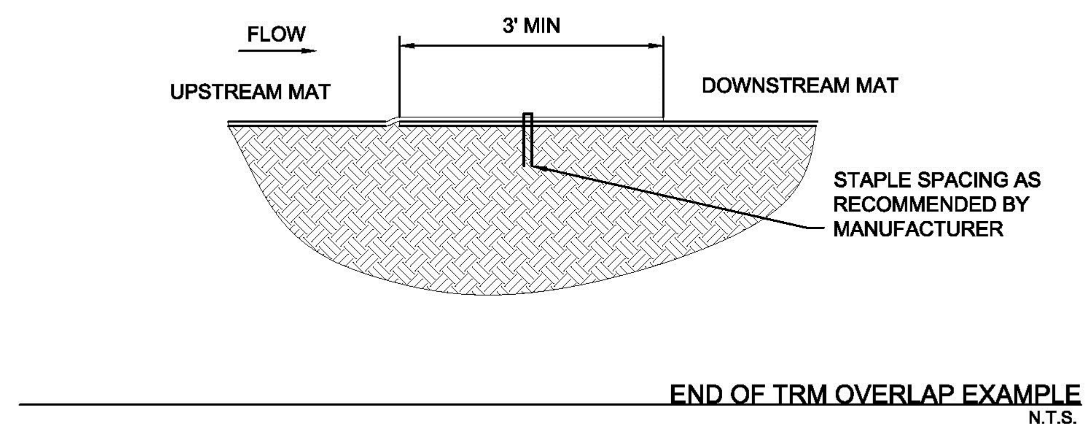
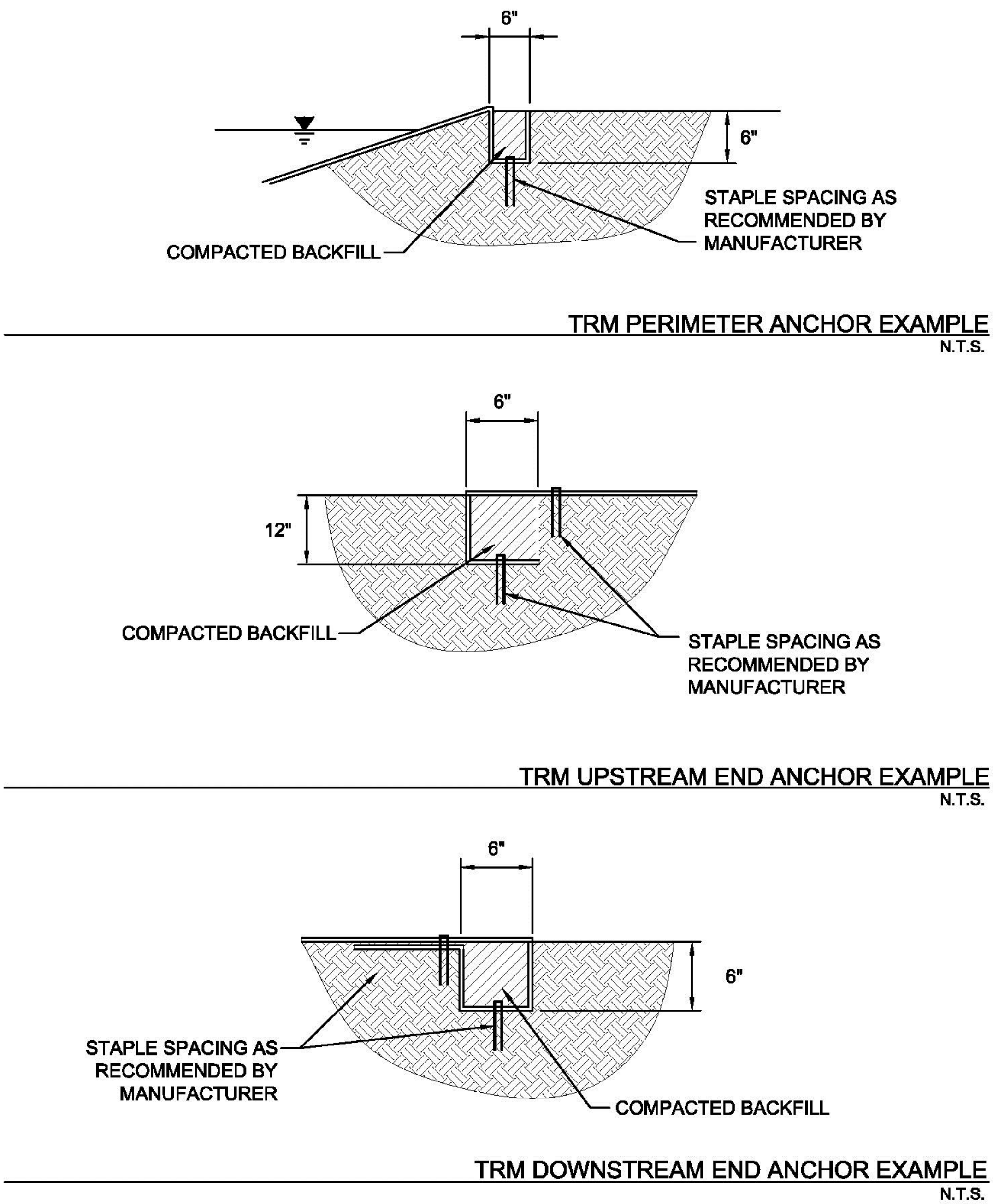


Figure 2.11 Schematics of Turf Reinforcement Mats
(Sources: Modified from American Excelsior Company and Texas Department of Transportation)

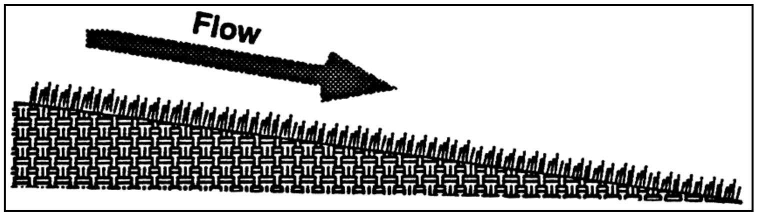


NOTE: ANCHORING METHODS PROVIDED ARE EXAMPLES OF THE TYPE OF ANCHORING THE TRM MANUFACTURER MAY RECOMMEND. THERE ARE DOZENS OF DIFFERENT ANCHORING METHODS DEPENDING ON THE TRM AND SITE-SPECIFIC CONDITIONS. ALWAYS FOLLOW THE MANUFACTURER'S RECOMMENDATIONS FOR ANCHORING.

Figure 2.12 Examples of Turf Reinforcement Mat Anchoring
(Source: Modified from Texas Department of Transportation Soil Retention Blanket Product Installation Sheet)

2.9 Vegetation

Erosion Control



Description: Vegetation, used as an erosion control, is the sowing or sodding of grasses, small grains, or legumes to provide temporary and final vegetative stabilization for disturbed areas.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Specify preparation of the soil surface before seeding or sodding
- Minimum of 4 to 6 inches of top soil required, depending on subsurface conditions
- Specify soil amendments depending on soil conditions
- Select seed or sod species appropriate for the climate, season, and soil

ADVANTAGES / BENEFITS:

- More effective and easier to maintain than sediment controls during a long construction period
- May be used for temporary or final stabilization

DISADVANTAGES / LIMITATIONS:

- Not appropriate for areas with heavy pedestrian, vehicular traffic, or concentrated, high velocity flow
- May require days to weeks for adequate establishment
- Alternate erosion control is needed until vegetation is established

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Protect newly seeded areas from excessive runoff, high velocity flow, and traffic until vegetation is established
- Water and fertilize until vegetation is established
- Reseed and/or provide mulch or another control for bare spots
- Rake accumulations of sediment from the vegetation

TARGETED POLLUTANTS

● Sediment

◒ Nutrients & Toxic Materials

○ Oil & Grease

○ Floatable Materials

○ Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.90

(*When fully established; lower while vegetation is first growing*)

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs

◒ Maintenance

○ Training

◒ Suitability for Slopes > 5%

Other Considerations:

- *Design is unique to soil and other conditions at each site*
- *Watering and other maintenance required until vegetation is established*

2.9.1 Primary Use

Vegetation is used as a temporary or final stabilization measure for areas disturbed by construction. As a temporary control, vegetation is used to stabilize stockpiles, earthen dikes, and barren areas that are inactive for longer than two weeks. As a final control at the end of construction, grasses and other vegetation provide good protection from erosion along with some filtering for overland runoff. Subjected to acceptable runoff velocities, vegetation can provide a positive method of long-term stormwater management as well as a visual amenity to the site.

Other control measures may be required to assist during the establishment of vegetation. These other controls include erosion control blankets, mulching, swales, and dikes to direct flow around newly seeded areas and proper grading to limit runoff velocities during construction.

2.9.2 Applications

Vegetation effectively reduces erosion in channels and swales and on stockpiles, dikes, and mild to medium slopes. Vegetative strips can provide some protection and sediment trapping when used as a perimeter control for utility and site development construction. Refer to *Section 3.15 Vegetated Filter Strips and Buffers* for more information.

In many cases, the initial cost of temporary seeding may be high compared to tarps or covers for stockpiles or other barren areas subject to erosion. This initial cost should be weighed with the amount of time the area is to remain inactive, since vegetation is more effective and the maintenance cost for vegetated areas is much less than most structural controls.

2.9.3 Design Criteria

General

- Vegetation is a highly effective erosion control when the vegetation is fully established. Until then, additional controls are needed. Sediment controls should not be removed from vegetated areas until the vegetation is established.
- On grades steeper than 20:1 (5 percent), anchored mulch or erosion control blankets are required to protect seeded areas until vegetation is established. Refer to *Section 2.5 Mulching* and *Section 2.3 Erosion Control Blankets* for design criteria.
- Vegetation may be used by itself for channel protection when the channel grade is less than 2 percent and the temporary control design storm (2-year, 24-hour) and the conveyance storm (25-year, 24-hour) flow velocities are less than 6 feet per second.
- If the velocity of the temporary control design storm is greater than 2 feet per second, erosion control blankets shall be used in the channel while vegetation is being established. Turf reinforcement mats are required when the velocity exceeds 6 feet per second. Refer to *Section 2.3 Erosion Control Blankets* and *Section 2.8 Turf Reinforcement Mats* for design criteria.
- Stabilization of channels with vegetation is limited to channels that have side slopes of 3:1 or flatter.
- On cut/fill slopes and channels designed to receive temporary or final vegetation, establishment of vegetation shall be initiated immediately after completing grading of the cut/fill slope or channel, and in no case later than 14 days after completion of grading on these features. It is not acceptable to delay establishing vegetation on these highly-erodible areas until completion of construction activities and stabilization of the remainder of the site.

Surface Preparation

- Unless infeasible, remove and stockpile existing topsoil at the start of grading activities. Store topsoil in a series of small stockpiles instead of one large stockpile to decrease the loss of aerobic soil micro-organisms during stockpiling.

- Interim or final grading must be completed prior to seeding or sodding.
- To minimize soil compaction of areas to be vegetated, limit vehicle and equipment traffic in these areas to the minimum necessary to accomplish grading.
- Install all necessary erosion structures such as dikes, swales, diversions, etc. prior to seeding or sodding.
- Spread stockpiled topsoil evenly over the disturbed area to be vegetated.
- Depth of topsoil shall be a minimum of 4 inches, with 6 inches required where the topsoil is over rock, gravel or otherwise unsuitable material for root growth. After spreading stockpiled topsoil, provide additional top soil as needed to achieve these depths.
- Compost Manufactured Topsoil as specified in TxDOT Special Specification 1001 may be used to achieve the specified depths or when it's infeasible to stockpile topsoil. Topsoil may also be acquired from another construction site if there is no space to stockpile the topsoil at that site.
- Topsoil shall have an organic content of 10 to 20 percent using ASTM D2974 Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils.
- Topsoil that does not meet the organic content requirement shall be amended with General Use Compost as specified in TxDOT Special Specification 1001. Amendment should be three parts of topsoil to one part compost by volume thoroughly blended.
- Seed bed should be well pulverized and loosened to a minimum depth of 3 inches and then raked to have a uniform surface.
- When establishing vegetation from seed, groove or furrow slopes steeper than 3:1 on the contour line before seeding.

Plant Selection, Fertilization and Seeding

- Use only high quality, USDA certified seed.
- Use an appropriate species or species mixture adapted to the local climate, onsite soil conditions and the season as shown below, or consult with the local office of the Natural Resource Conservation Service (NRCS) or Texas AgriLife Extension Service for selection of proper species and application technique in this area.
- Seeding rate should be in accordance with the Tables 2.4, 2.5 and 2.6 as follow in this section or as recommended by the Natural Resources Conservation Service (NRCS) or Texas AgriLife Extension Service.
- Chemical fertilization is not recommended at the time of seeding, because it typically stimulates and is consumed by fast growing weeds that out-compete the slower growing grasses and legumes. If the topsoil has not been amended by compost as discussed above, an 0.5 inch layer of General Use Compost (TxDOT Special Specification 1001) is recommended as a surface treatment to protect the seed and provide slow release nutrients
- Evenly apply seed using a seed drill, cultipacker, terraseeding, or hydroseeder.
- Hydro-seeding should not be used on slopes of 5:1 or steeper unless Bonded Fiber Matrix is used.
- Seeded areas shall be thoroughly watered immediately after planting. Water shall be applied at a rate that moistens the top 6 inches of soil without causing runoff. Provide water daily for the first 14 days after seeding and thereafter as needed to aid in establishment of vegetation.
- Use appropriate mulching techniques (*Section 2.5 Mulching*) where necessary, especially during cold periods of the year. Mulch consisting of chipped site vegetation is discouraged, since the wood content may result in depleting nitrogen from the soil.

Sodding

- Use of sod should be limited to planned landscapes due to the relatively high water use of most types of sod grass.
- When sod is necessary to achieve immediate stabilization, buffalograss (*Buchloe dactyloides*) is recommended. Other types of sod may be used in landscaping when specified by a landscape architect for a commercial property or a homebuyer for a residential lot.
- The sod should be mowed prior to sod cutting so that the height of the grass shall not exceed 3 inches and should not be harvested or planted when its moisture condition is so excessively wet or dry that its survival shall be affected.
- Sod shall have a healthy, virile, system of dense, thickly matted roots throughout a minimum soil thickness of 0.75 inch.
- Sod shall be planted within 3 days after it is excavated.
- In areas subject to direct sunlight, pre-moisten prepared sod bed by watering immediately prior to placing sod.
- Sodded areas shall be thoroughly watered immediately after they are planted.

Temporary Vegetation

The following table lists recommended plant species for the North Central Texas region depending on the season for planting.

Table 2.4 Recommended Grass Mixture for Temporary Erosion Control

Season	*Common Name*	*Pure Live Seed Rate (Lbs/Acre)*
Sep 1 - Nov 30	Tall Fescue Western Wheat Grass Wheat (Red, Winter)	4.5 5.6 34.0
May 1 - Aug 31	Foxtail Millet	34.0
Feb 15 – May 31 Sep 1 – Dec 31	Annual Rye	20.0

Areas receiving temporary seeding and vegetation shall be landscaped, re-seeded or sodded with perennial species to establish final vegetation at the end of construction.

Vegetation for Final Stabilization

Sodding or seeding may be used to establish vegetation for final stabilization of areas disturbed by construction activity. The vegetation must achieve a cover that is 70 percent of the native background vegetative cover to be considered final stabilization. Sod will achieve this coverage quicker than seeding; however, sod is usually more expensive than seeding. Sod is most cost-effective for small areas or areas of concentrated flow or heavy pedestrian traffic where it will be difficult to establish vegetation by seeding.

Grass seed for establishing final stabilization can be sown at the same time as seeding for temporary (annual) vegetation. Drought tolerant native vegetation is recommended rather than exotics as a long-term water conservation measure. Native grasses can be planted as seed or placed as sod. Buffalo 609, for example, is a hybrid grass that is placed as sod. Fertilizers are not normally used to establish native grasses, but mulching is effective in retaining soil moisture for the native plants.

Table 2.5 Recommended Grass Mixture for Final Stabilization of Upland in Rural Areas					
County	***Planting Date***	***Clay Soils***		***Sandy Soils***	
		Species and Pure Live Seed Rate (Lbs/Acre)		***Species and Pure Live Seed Rate (Lbs/Acre)***	
Erath Hood Johnson Palo Pinto Parker Somervell Tarrant Wise	February 1 – May 15	Green Sprangletop Sideoats Grama (El Reno) Bermudagrass Little Bluestem (Native) Blue Grama (Hachita) Illinois Bundleflower	0.3 2.7 0.9 1.0 0.9 1.0	Green Sprangletop Sand Lovegrass Bermudagrass Weeping Lovegrass (Ermelo) Sand Dropseed Partridge Peal	0.3 0.5 1.8 0.8 0.4 1.0
Collin Dallas Denton Ellis Kaufman Navarro Rockwell	February 1 – May 15	Green Sprangletop Bermudagrass Sideoats Grama (El Reno) Little Bluestem (Native) Buffalograss (Texoka) Illinois Bundleflower	0.3 1.2 2.7 2.0 1.6 1.0	Green Sprangletop Bermudagrass Weeping Lovegrass (Ermelo) Sand Lovegrass Sand Dropseed Partridge Pea	0.3 1.8 0.6 0.6 0.4 1.0
Hunt	February 1 – May 15	Green Sprangletop Sideoats Grama (El Reno) Bermudagrass Little Bluestem (Native) Illinois Bundleflower	0.3 3.2 1.8 1.7 1.0	Green Sprangletop Bermudagrass Bahiagrass (Pensacola) Sand Lovegrass Weeping Lovegrass (Ermelo) Partridge Pea	0.3 1.5 6.0 0.6 0.8 1.0

(Source: TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 164)

Table 2.6 Recommended Grass Mixture for Final Stabilization of Upland in Urban Areas					
County	***Planting Date***	***Clay Soils***		***Sandy Soils***	
		Species and Pure Live Seed Rate (Lbs/Acre)		***Species and Pure Live Seed Rate (Lbs/Acre)***	
Erath Hood Johnson Palo Pinto Parker Somervell Tarrant Wise	February 1 – May 15	Green Sprangletop Sideoats Grama (El Reno) Bermudagrass Buffalograss (Texoka)	0.3 3.6 2.4 1.6	Green Sprangletop Sideoats Grama (El Reno) Bermudagrass Sand Dropseed	0.3 3.6 2.1 0.3
Collin Dallas Denton Ellis Kaufman Navarro Rockwell	February 1 – May 15	Green Sprangletop Sideoats Grama (El Reno) Buffalograss (Texoka) Bermudagrass	0.3 3.6 1.6 2.4	Green Sprangletop Buffalograss (Texoka) Bermudagrass Sand Dropseed	0.3 1.6 3.6 0.4
Hunt	February 1 – May 15	Green Sprangletop Bermudagrass Sideoats Grama (Haskell)	0.3 2.4 4.5	Green Sprangletop Bermudagrass	0.3 5.4

(Source: TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 164)

Vegetation for final stabilization of channels requires grasses that are tolerant of periodic inundation, such as Bermuda grass, Kentucky bluegrass or a grass-legume mixture.

Additional Considerations

- Conditions for establishing vegetation vary significantly from site to site. Therefore, specifics of the vegetation design should be prepared based on the soil, slopes, drainage patterns, and the purpose of the vegetation at a each site.
- For construction activities that include landscaping in the development plans, the landscape architect should be consulted when specifying vegetation for temporary or final stabilization of disturbed areas.
- Vegetation is easier to establish if equipment and vehicle traffic is managed onsite to minimize soil compaction by traffic in the disturbed area that will be vegetated.
- Establishing a good vegetative cover is dependent on the season of the year. Projects that commence in the fall of the year may not be candidates for using vegetation as an erosion control.
- Where vegetation is used in swales and channels it may be necessary to use sod, rather than seeding, to establish an erosion resistant surface that accommodates rainfall runoff flows.
- Mulch should be used to enhance vegetative growth, in that mulch protects seeds from heat, prevents soil moisture loss, and provides erosion protection until the vegetation is established. Compost mulch has the additional benefit of providing some slow-release nutrients.
- Fertilizers have both beneficial and adverse effects. Fertilizers provide nutrients to the vegetation, but fertilizers are also a source of unwanted nutrients in streams and lakes. In this latter regard, they are a pollutant. The use of native vegetation rather than exotics reduces the need for fertilizers. Organic fertilizers, such as compost mulch, are generally preferred over chemical fertilizers. They provide a slow release of nutrients over a longer period of time and are less likely to cause environmental problems.
- Steep slopes represent a problem for establishing vegetation. Hydraulic mulches are useful for establishing vegetation on slopes. Refer to *Section 2.5 Mulching*.

2.9.4 Design Guidance and Specifications

Additional criteria for the application of vegetation in channels are in *Section 3.6.3 of the iSWM Criteria Manual* and design guidance is in *Section 3.2 of the Hydraulics Technical Manual*.

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Item 202 Landscaping. Additional specifications for the following components of this item are in the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT 2004):

- Topsoil, Item 160.
- Compost, Item 161.
- Sodding for Erosion Control, Item 162.
- Seeding for Erosion Control, Item 163.
- Fertilization, Item 164.
- Vegetative Watering 165.

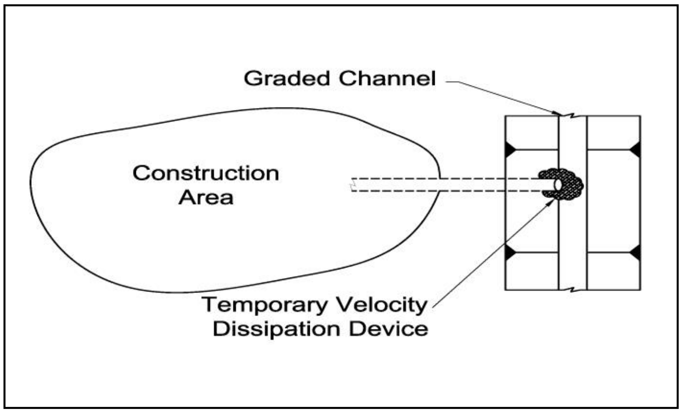
2.9.5 Inspection and Maintenance Requirements

Protect newly seeded areas from excessive runoff and traffic until vegetation is established. Include a watering and fertilizing schedule in the iSWM Construction Plan facilitate the establishment of the vegetation. Vegetation for final stabilization must be maintained until the vegetative cover is 70 percent of the native background vegetative cover.

Vegetation should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to ensure that the plant material is established properly and remains healthy. Bare spots shall be reseeded and/or protected from erosion by mulch or other measures. Accumulated sediment deposited by runoff should be removed to prevent smothering of the vegetation. In addition, determine the source of excess sediment and implement appropriate measures to control the erosion.

2.10 Velocity Dissipation Devices

Erosion Control



Description: Velocity dissipation devices control erosion by dispersing concentrated flow and slowing flow velocities at drainage pipe outlets, the outlet end of an armored flume or swale, and other points where concentrated flow is discharged to an open channel. Velocity dissipation devices are also called energy dissipaters. They may consist of crushed rock, rock riprap, gabions, and other non-erosive materials.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Use at discharge points into unlined and natural channels where the flow velocity exceeds 4 fps during construction
- Install permanent energy dissipaters in the first phase of construction when possible to eliminate the need for temporary devices
- Design based on discharge rate and velocity for the temporary control design storm (2-year, 24-hour)

ADVANTAGES / BENEFITS:

- Protects habitat in natural channels
- Protects new conveyance systems from damage due to erosion until permanent controls are installed

DISADVANTAGES / LIMITATIONS:

- Additional cost for temporary structures
- May be damaged by larger storm events

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Repair damaged devices and eroded areas
- Replace dislodged rock

TARGETED POLLUTANTS

○ Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=N/A

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
○ Maintenance
○ Training
◒ Suitability for Slopes > 5%

Other Considerations:

- *Coordination of temporary structures with the plans for permanent infrastructure*

2.10.1 Primary Use

Velocity dissipation devices are used to disperse concentrated flow and slow velocities to a point where they will not cause erosion in a vegetated or natural drainage way. In process of slowing the flow, suspended sediments in runoff from disturbed areas may be removed from the runoff and settle in the dissipation device.

2.10.2 Applications

Velocity dissipation devices are used where velocities in concentrated flow may cause erosion of un-lined or natural channels during construction. These locations are typically where a constructed conveyance system (such as a storm drain pipe, concrete flume, or roadside drainage ditch) discharges flow to a channel that is larger in size or lower in elevation.

2.10.3 Design Criteria

General

- Temporary velocity dissipation devices should be installed at pipe outlets and similar discharge points during construction to maintain the downstream physical and biological characteristics and functions until channel protection and stabilization measures are installed. Other points that may require velocity dissipaters are locations where concrete flumes, drainage swales, roadside ditches, and other drainage structures discharge to an unlined or natural channel.
- The design and use of velocity dissipation devices during construction should be coordinated with the stormwater infrastructure design in the development plans. It is recommended that permanent devices be constructed early in the first phase of construction to provide velocity dissipation both during and post-construction, thus eliminating the need for temporary devices.
- The criteria in this section are specific to **temporary** velocity dissipation devices that are designed using the temporary control design storm (2-year, 24-hour). The design of permanent dissipation devices shall be in accordance with the municipality's drainage design criteria and are more stringent.
- Temporary dissipation devices must not block flow or cause flooding during larger storm events.
- Temporary dissipation devices shall be installed on all outlets where the design storm velocity exceeds 4 feet per second and the discharge is to an unlined or natural channel.

Rock Riprap

- Rock riprap is the most common material used for temporary velocity dissipation. The rock may be removed and re-used for other applications when permanent drainage structures, channel lining, or final stabilization measures are installed.
- Design calculations are required for the use of this control. The designer shall provide drainage computations, discharge velocity, stone size, and apron dimensions for each application.
- Rock may be natural stone or recycled concrete.
- The stone shall be well graded from 2 inch diameter through the median diameter (d_{50}) and up to the maximum diameter (d_{MAX}). The stone should create a homogeneous stone surface with no voids larger than 1½ inches in diameter.
- Stone shall be sized using the criteria for riprap aprons in *Section 4.0 of the Hydraulics Technical Manual* or using an alternative method accepted by the municipality reviewing the plans. The median stone size (d_{50}) shall be a minimum of 6 inches for temporary velocity dissipation. The maximum stone size (d_{MAX}) shall be 1.5 times d_{50}.
- Minimum depth of the riprap apron shall be 1.5 times d_{MAX}.

- Minimum length of the apron shall be 4.5 times the outlet pipe diameter or equivalent for other types of outlets.
- Minimum width of the apron shall be 4.0 times the outlet pipe diameter or equivalent for other types of outlets.
- Riprap should be placed on a lining of filter fabric to prevent soil movement into or through the riprap. The perimeter of the filter fabric must be keyed into the ground a minimum of 6 inches.
- Riprap apron should be aligned with flow direction.
- Riprap shall not be used where there is a difference in elevation between the outlet and the receiving channel.

Other Devices

- Articulating concrete blocks, gabions, stilling basins or manufactured velocity dissipaters may be used for velocity dissipation if the designer provides calculations that document size and dimensions of the device for the design storm flow rate and velocities.
- Temporary baffled chutes, gabion drop structures, or other stabilized grade breaks shall be installed where an elevation difference exists at the outlet until permanent structures are installed.

2.10.4 Design Guidance and Specifications

Criteria for the design of permanent design velocity dissipation devices are in *Section 3.6.3 of the iSWM Criteria Manual*, and additional design guidance is in *Section 4.0 of the Hydraulics Technical Manual*. Guidance is also available in the Federal Highway Administration Engineering Circular No. 14, Hydraulic Design of Energy Dissipaters for Culverts and Channels.

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Item 803, Slope and Channel Protection.

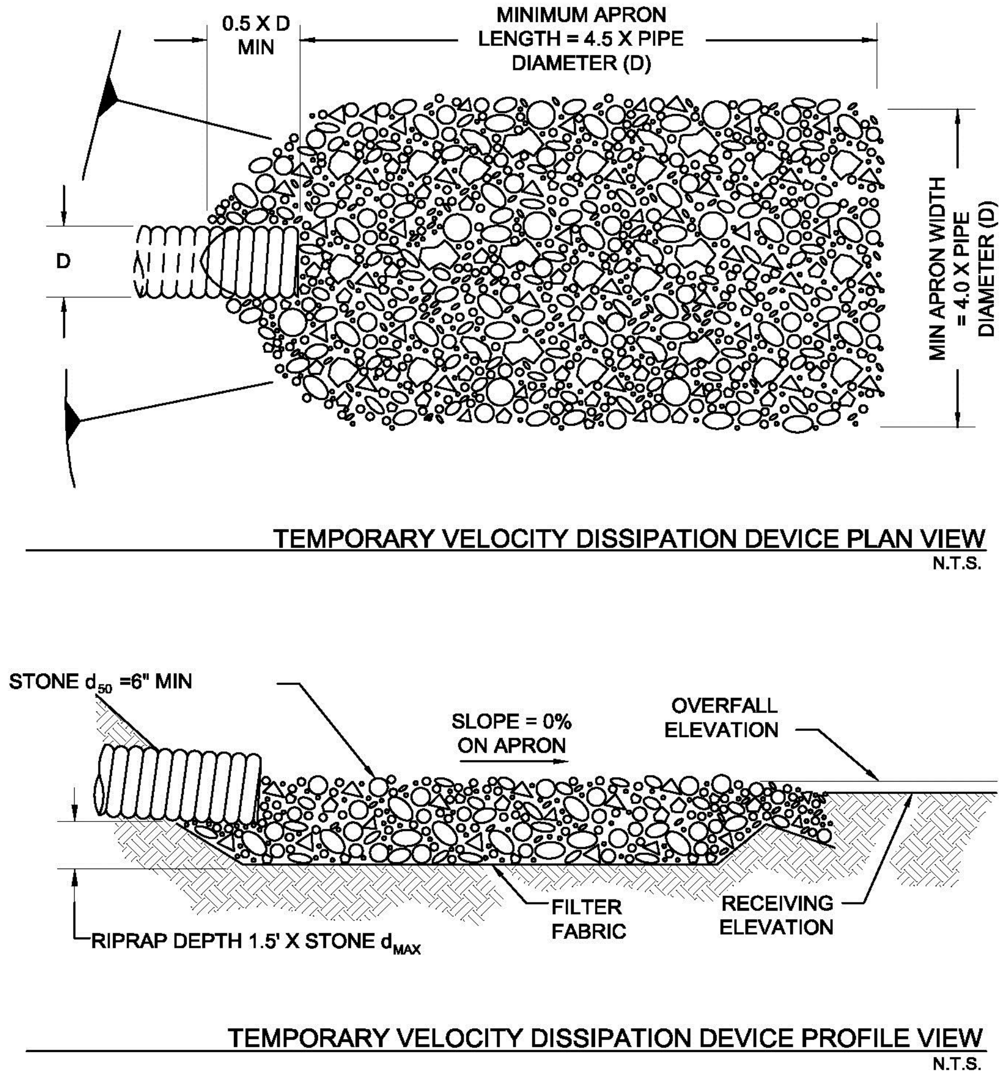
2.10.5 Inspection and Maintenance Requirements

Discharge points shall be inspected regularly (at least as often as required by the TPDES Construction General Permit) for evidence of downstream erosion. Repair dislodges or missing rock riprap. The development of head-cuts, the deepening or widening of the channel, or low flow channels developing within the main channel are all evidence that additional velocity dissipation measures are required until permanent structures are installed.

2.10.6 Example Schematics

The following schematics are only applicable to **temporary** installations of riprap for velocity dissipation. Permanent installations shall be in accordance with the municipality's design criteria.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



NOTE: DIMENSIONS OF THE RIPRAP APRON SHALL BE DESIGNED BASED ON FLOW CONDITIONS. TEMPORARY CONTROL DESIGN STORM (2-YEARS, 24-HOUR). PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE APRON.

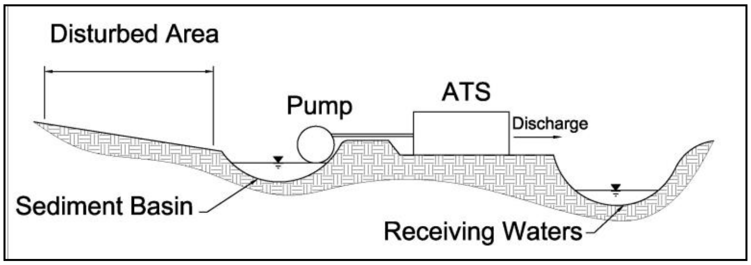
- PIPE DIAMETER (OR EQUIVALENT FOR FLUME, SWALE, ETC.)
- DISCHARGE VELOCITY FROM DRAINAGE STRUCTURE
- MEDIAN STONE DIAMETER (d_{50}) AND MAXIMUM STONE DIAMETER (d_{MAX})
- DEPTH OF APRON

Figure 2.13 Schematics of Velocity Dissipation Device
(Source: Modified from Oklahoma City Public Works Engineering Division Detail ERO-A17)

3.0 Sediment Controls

3.1 Active Treatment System (ATS)

Sediment Control



Description: An Active Treatment System (ATS) is a small, onsite, water treatment plant used to produce discharge water quality that is better than can be achieved by traditional sediment controls. Common ATS methods are filtration and chemical-aided coagulation/flocculation.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- ATS designed based on site conditions, stormwater characteristics, and required discharge quality
- ATS shall be designed in coordination with the system provider and operator

ADVANTAGES / BENEFITS:

- Provides consistent, high quality, stormwater discharges

DISADVANTAGES / LIMITATIONS:

- Most expensive treatment method
- Retention structures required to capture the design storm for treatment
- Filtration requires pre-treatment with a sediment trap or basin
- Highly dependent on operator knowledge and skill level

MAINTENANCE REQUIREMENTS:

- Daily monitoring and maintenance while in operation, including influent characteristics and chemical dosage
- Backwash filters and dispose of waste from backwashing
- Monitor discharge for residual chemicals
- Repair erosion or other damage on stormwater retention structures that precede the ATS

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ◒ Oil & Grease
- ● Floatable Materials
- ● Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

$\boxed{\textbf{Sediment Barrier}}$

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.99

IMPLEMENTATION CONSIDERATIONS

- ● Capital Costs
- ● Maintenance
- ● Training
- ● Suitability for Slopes > 5%

Other Considerations:

- *Operator training*
- *Site access to operate the ATS during wet conditions*
- *Worker safety and spill response procedures*

3.1.1 Primary Use

Active treatment systems (ATS) are used when traditional sediment controls cannot achieve the necessary level of sediment removal for discharges from a construction site. They are primarily used to remove fine silt and clay soil particles, for which traditional sediment controls are the least effective. These fine particles are small enough to pass through the pores or void spaces of sediment barriers. They are also not removed by sediment basins, because their settling velocities require a detention time of days or weeks, not hours.

3.1.2 Applications

Active treatment systems are applicable on sites that have a large percentage of fine silt and clay soils. The systems are most useful where special aquatic sites or sensitive receiving waters result in specific limits on discharges or regulations require a higher level of treatment. An ATS may be used when a turbidity effluent limit is established for a construction activity or where the activity discharges to:

- Wetlands regulated under Section 404 of the Clean Water Act;
- Spring-fed receiving waters;
- Receiving water with a Total Maximum Daily Load;
- Receiving water bodies with a Water Quality Standard that could be exceeded by the discharge; or
- Receiving water utilized by a species protected under the Federal Endangered Species Act or the State of Texas Threatened and Endangered Species Regulations.

3.1.3 Design Criteria

Active treatment systems are a specialized application that requires skill in designing, operating, and maintaining the systems. When the designer has determined that an ATS is needed for a project, the designer should select a supplier of ATSs and work with their technical experts. The criteria contained in this section are general guidelines. It is essential that the designer of controls for a construction activity work with an ATS supplier and operator to develop an effective system based on site conditions and anticipated characteristics of the stormwater runoff.

General

- A source of electricity is required for an ATS. Diesel generators are required until the electrical distribution system is extended to the site. In some cases, it may be advisable to maintain the generators onsite for the duration of the project in case of power outages.
- An ATS requires a sediment basin, tank, or other structure to capture the temporary control design storm (2-year, 24-hour) and retain it to be pumped to the ATS. The retention structure should be designed to pass larger storm events without damage to the structure.
- An ATS can be either a batch flow or flow-through (continuous flow) design.
- ATS designs are specific to each site, the stormwater runoff characteristics, and the required discharge water quality. The designer should consult with suppliers and operators of ATSs and consider the following when designing the ATS:
    - Available stormwater detention space for the storm event being treated and for another event that could occur during treatment.
    - Turbidity, pH, and suspended solids concentrations of the stormwater to be treated.
    - Size of soil particles to be removed.
    - Required discharge concentrations.
    - Flow rate through the ATS.

   - Available space.
   - Cost.
- Electrocoagulation is available as an ATS for sediment removal; however, it is not recommended for construction sites.
- The design should include requirements for operator training and/or required skill and experience for the lead operator. Unlike other sediment control devices, improper operation can result in a discharge that is more damaging to the receiving water than the construction activity. The recommended minimum skill level is 5 years experience operating stormwater ATSs or a combination of training and experience equivalent to a Class C Surface Water Operators license in the State of Texas.
- The ATS operator selected for the project shall have written plans for the following:
   - Operation and maintenance manual for all equipment in the ATS.
   - Monitoring, sampling and reporting, including Quality Assurance/Quality Control (QA/QC).
   - Worker health and safety.
   - Spill prevention and response.
- The ATS shall be equipped with instrumentation that automatically measures and records the following:
   - Influent and effluent turbidity.
   - Influent and effluent pH.
   - Influent and effluent flow rate.
- The ATS should be designed with a recirculation mode or a safe shut down mode that will be automatically activated upon system upset, power failure, or other catastrophic event.
- A velocity dissipation device is required at the ATS discharge point.

Filtration

- Filtration is accomplished by pumping water through vessels filled with granular filter media. The media may be sand, gravel, anthracite or a combination. Single media, sand filters are most common in construction applications.
- Bag or cartridge filters may be used after the media filter to provide the highest level of sediment removal. They are typically only needed when extremely low turbidity values (<10 NTU) are required for discharges to clear, cool-water streams, such as spring-feed creeks flowing over a limestone channel bed.
- For temporary installations at construction sites, filtration units are frequently hauled to the site and operated on flat bed trailers.
- The designer shall specify the filter media to be used based on the particle size to be removed and desired reduction in turbidity and suspended solids concentrations.
- Filtration can be effective in removing other pollutants from construction sites, such as sheen on stormwater surfaces; however, the filter media must then be classified and handled as the appropriate type of waste.
- Filtration may be used as an ATS by itself on sites where the suspend soils are primarily coarser silts and sands and a higher quality discharge is required than can be achieved by traditional sediment controls.
- Filtration systems are most commonly used after chemical-aided flocculation to remove flocs that do not settle within the detention time available while maintaining the design flow rate.

- When used without chemical-aided flocculation, stormwater requires pre-treatment with a sediment trap or basin before being pumped to the filter. Pre-treatment extends the operating life of the filter and decreases maintenance requirements.
- Filters shall be equipped with gauges to measure differential pressure across the filter to monitor filter loading.
- Filtration designs shall contain a means for backwashing the filters and collection and disposal of the backwash water.

Chemical-Aided Flocculation

- Chemicals are added as coagulation agents in an ATS. The coagulants destabilize the charged soil particles. As a result, the particles form flocs that can be settled or filtered from the stormwater.
- The ATS typically consists of the following, each of which requires its own design parameters:
  - Retention basin or other structure to capture the design storm.
  - Water pump to convey stormwater from the retention structure to the settling tank.
  - Chemical injection and metering pump.
  - Settling tank or chamber.
  - Filters (optional).
- Commonly used chemicals for stormwater treatment are chitosan, polyacrylamide (PAM), aluminum sulfate (alum), and polyaluminum chloride.
- Chemicals must be applied in proper doses and for the proper contact times to avoid potential toxicity in the ATS effluent. The effluent should be monitored for both turbidity and residual concentration of the treatment chemical.
- Where feasible, chemical injection should occur on the intake side of the stormwater pump to provide for maximum mixing.
- Chemical dosing should be designed based on flow rate, pH, and suspended solids concentration. Adjustments to dosage are common as the stormwater characteristics vary for different storm events and changing conditions on the construction site.
- Jar tests should be used to determine the chemical dosage. Jar tests should be conducted in accordance with ASTM D2035 Standard Practice for Coagulation-Flocculation Jar Test of Water. Tests shall be performed 15 minutes after start-up and every 8 hours of operation.
- The settling tank or chamber should be designed to prevent the accidental discharge of settled floc during floc pumping and related cleaning operations. Include specifications for disposal of settled floc.
- When chitosan is used, the discharge from the ATS should be tested for residual concentration of the chemical using commercially available residual field tests. Tests should be performed 15 minutes after start-up, every 8 hours of operation, and 15 minutes after each change in dosage. Return period of the test results depends on the sensitivity of the receiving water, but in no case should be longer than 24 hours. Return period may be as short as one hour if the receiving water has a species that is threatened, endangered, or of concern.
- The residual concentration of chitosan should be limited to 10 percent or less of the following for the aquatic species most sensitive to the chemical being used:
  - Geometric mean of the No Observed Effect Concentration (NOEC).
  - Acute toxicity concentration.
  - Chronic toxicity concentration.

- For PAM and other coagulation agents without a residual field test, a daily bioassay shall be performed on an effluent sample. The methods used for acute toxicity testing shall be those outlined for a 96-hour acute test in Methods for Measuring the Acute Toxicity of Effluents and Receiving Water to Freshwater and Marine Organisms (USEPA-841-R-02-012) for Fathead minnow, *Pimephales promelas.*
- PAM has a documented record of low toxicity. For all other chemical coagulants without a residual field test, batch operation of the ATS is encouraged to delay discharge of the treated stormwater until results of the toxicity tests are available.
- Toxicity testing should be done by an independent, third-party laboratory that is accredited in Texas according to the standards of the National Environmental Laboratory Accreditation Conference (NELAC).

3.1.4 Design Guidance and Specifications

No specification for construction of this item is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.1.5 Inspection and Maintenance Requirements

Active treatment systems must be maintained and monitored by trained, onsite personnel that observe the system at all times while it is in operation. Inspection and maintenance should be according to the system's operations and maintenance manual.

The overall system should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to ensure stormwater is not bypassing the ATS. The basin or other structure used to collect and pre-treat stormwater should be inspected for damage and repaired as needed.

During operation of chemical-aided flocculation, the chemical dosage should be monitored and changed according to characteristic of the stormwater inflow. The discharge from the ATS should be sampled and analyzed regularly to verify that chemical residuals are acceptable levels.

3.1.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

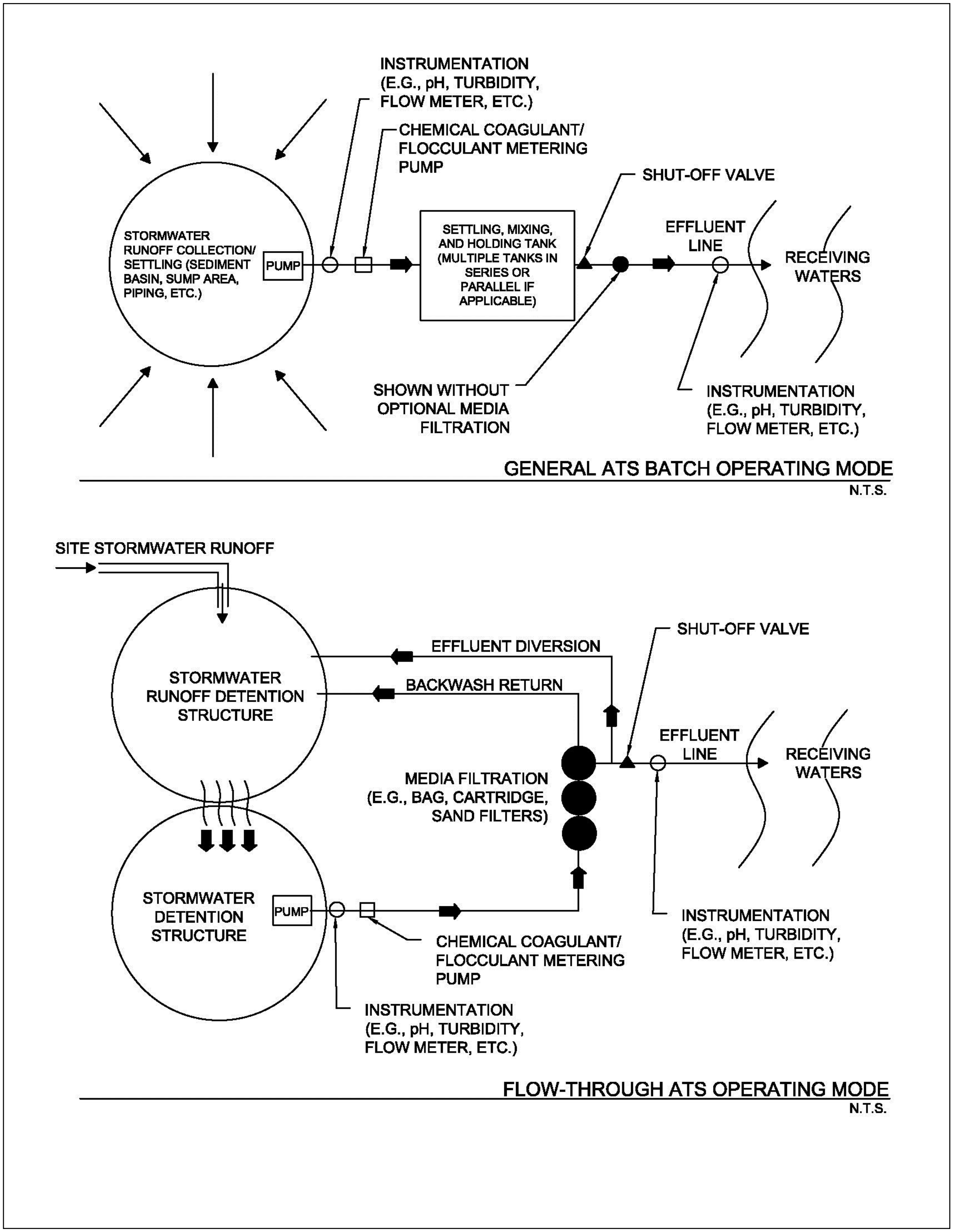


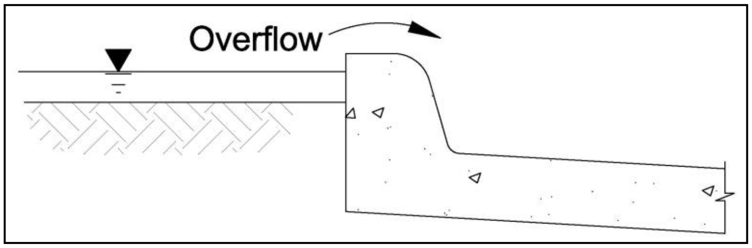
Figure 3.1 Schematics of Active Treatment System

(Source: EPA Development Document for Final Effluent Guidelines and Standards for the Construction & Development Category)

3.2 Depressed Grade (Curb Cut-Back) Sediment Trap

(Source: Modified from City of Plano BMP SP-12)

Sediment Control



Description: A depressed grade sediment trap is a sediment barrier created by grading or leaving the grade of an area at the back of curb or edge of pavement depressed to detain the surface flow until overflows onto the pavement.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum 4 feet width and 1.5 inch depth
- Maximum 2 percent longitudinal slope and 3 percent transverse slope
- Erosion control blankets required at low point (sag) curb inlets

ADVANTAGES / BENEFITS:

- Inexpensive sediment trap for very small areas
- Alternative to inlet protection for projects within rights-of-way
- May be used on individual residential lots in certain situations

DISADVANTAGES / LIMITATIONS:

- May be disturbed and altered by construction equipment driving through it
- Limited application to very small areas along rights-of-way and residential lots

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Use a shovel or blade to remove sediment
- Re-grade as necessary
- Inspect erosion control blankets and repair as needed

TARGETED POLLUTANTS

● Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control (boxed)
Slope Protection
Sediment Barrier (boxed)
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.50-0.75
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

○ Capital Costs
◒ Maintenance
○ Training
○ Suitability for Slopes > 5%

Other Considerations:

- *None*

3.2.1 Primary Use

Depressed grade sediment traps are used to intercept and trap flows from very small drainage areas (i.e. parkways, medians, and pavements).

3.2.2 Applications

Depressed grade sediment traps are used at construction sites within rights-of-way to control small drainage areas. It can be used at the back of curb or edge of pavement where the drainage area is limited to the parkway or median. It can also be used where sections of pavement are removed and replaced for pavement repair or underground utility installation.

3.2.3 Design Criteria

- The width of the excavated area when installed back of curb shall be a minimum of 4 feet.
- The longitudinal slope along the back of curb depression cannot exceed 2 percent and the transverse slope toward the back of curb cannot exceed 3 percent. Steeper slopes require additional sediment controls.
- The maximum width of the right-of-way draining into the sediment trap shall be 11.5 feet. No other drainage area may contribute runoff to the sediment trap.
- The depressed grade sediment trap may be used back of curb for sediment control on single residential lots if no other drainage area contributes runoff to the depressed area. The designer shall calculate the minimum width of the depressed area, based on a 1.5 inch depth, the length of the curb at the front of the lot, and the volume of runoff from the lot for the temporary control design storm (2-year, 24-hour).
- Erosion control blankets (ECBs) are required at low or sag points along the curb where flow may become more concentrated. Criteria for ECBs are in *Section 2.3 Erosion Control Blankets*.
- The excavation of the cut may be offset a maximum distance of 5 feet from the curb to avoid utility boxes.
- When a curb cut for a driveway is encountered and no driveway has been constructed, securely install a plank of wood (2x4, 4x4) across the cur cut in order to continue the curb.

3.2.4 Design Guidance and Specifications

No specification for depressed grade sediment trap is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.2.5 Inspection and Maintenance Requirements

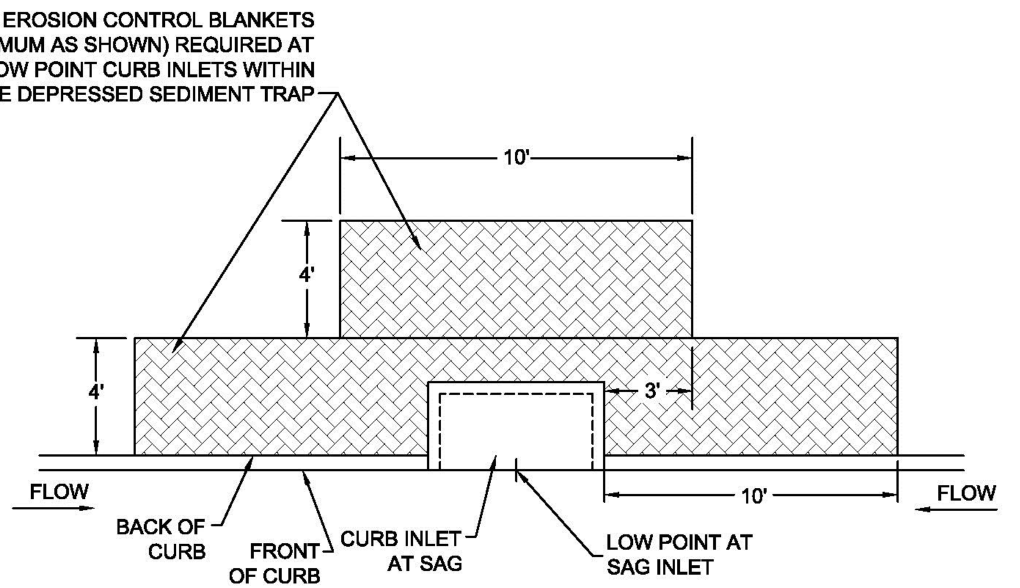
Depressed grade sediment traps should be inspected regularly (at least as often as required by TPDES Construction General Permit). Inspect the depression area periodically to ensure that the necessary storage volume is available. Use a shovel or blade to remove sediment from the area back of curb as needed. Re-grade the depression if it's disturbed by construction traffic.

The low points where this method is used should also be monitored during rain events to ensure the erosion control blankets are adequate to prevent sediment from flowing onto the pavement. Additional controls shall be added as needed.

3.2.6 Example Schematics

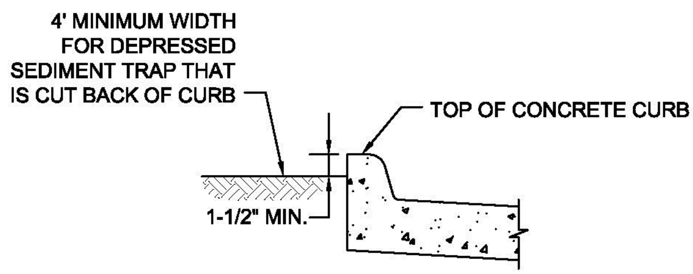
The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



DEPRESSED BACK OF CURB SEDIMENT TRAP PLAN VIEW

N.T.S.



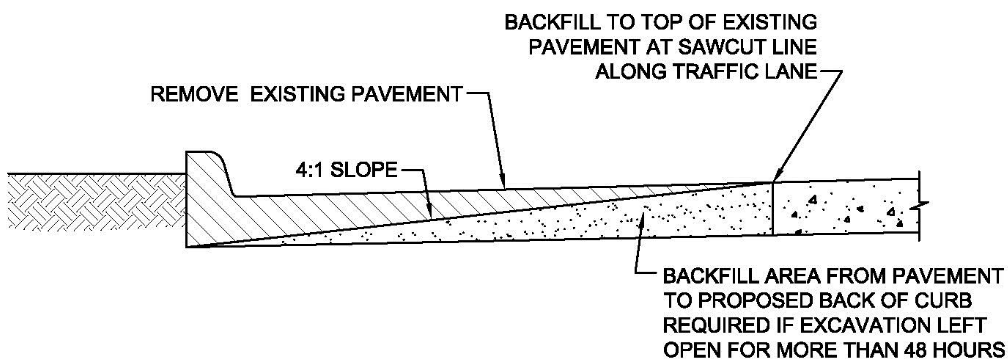
DEPRESSED BACK OF CURB SEDIMENT TRAP CROSS SECTION

N.T.S.

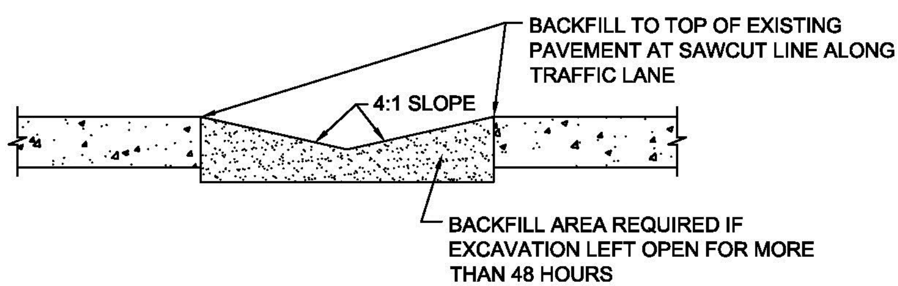
NOTES:
1. THE DEPRESSED SEDIMENT TRAP MAY BE OFFSET A MAXIMUM DISTANCE OF 5 FEET FROM THE CURB IF NECESSARY TO AVOID UTILITY BOXES.
2. WHEN A CURB CUT FOR A FUTURE DRIVEWAY IS ENCOUNTERED, INSTALL 2" X 4" BOARDS ACROSS THE CURB TO FORM A TEMPORARY "CURB" FOR THE SEDIMENT TRAP.

Figure 3.2 Schematics of Depressed Grade (Curb Cut-Back) Sediment Trap

(Source: City of Plano BMP SP-12)



DEPRESSED PAVEMENT REPLACEMENT SEDIMENT TRAP AT CURB
N.T.S.

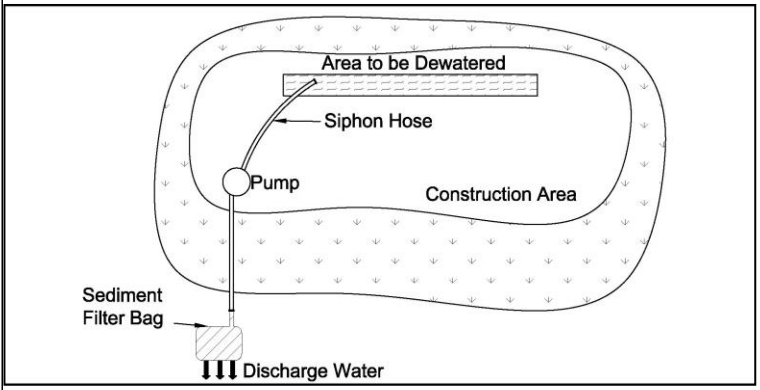


DEPRESSED PAVEMENT REPLACEMENT SEDIMENT TRAP AT TRENCH
N.T.S.

Figure 3.3 Schematics of Depressed Pavement Replacement Sediment Trap
(Source: City of Plano BMP SP-12)

3.3 Dewatering Controls

Sediment Control



Description: Dewatering controls consist of methods and devices to remove suspended soil in water that is pumped or otherwise discharged from foundations, trenches, excavations, and other low areas. The controls may be the sediment controls already onsite (e.g. silt fence, organic filters tubes) or dedicated dewatering devices such as sediment tanks and sediment filter bags.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Discharge of pumped water is prohibited unless controlled for the removal of suspended soil
- Select controls based on dewatering flow rate and duration and available space
- Dewatering discharge points must be protected for high velocities

ADVANTAGES / BENEFITS:

- Removes suspended soil and some pollutants from pumped water
- Works well with passive treatment systems for removal of clay soil particles
- Water may be applied to other onsite uses

DISADVANTAGES / LIMITATIONS:

- Requires frequent maintenance

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Monitor for erosion, control failure and unauthorized discharges frequently while pumping
- Clean and replace controls as they are filled with sediment

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.50-0.75
(*Depends on soil type*)

TARGETED POLLUTANTS

● Sediment

○ Nutrients & Toxic Materials

◒ Oil & Grease

○ Floatable Materials

○ Other Construction Wastes

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs

● Maintenance

◒ Training

○ Suitability for Slopes > 5%

Other Considerations:

- *None*

3.3.1 Primary Use

Dewatering controls are used to remove suspended soil in water that is pumped or otherwise discharged from foundations, trenches, excavations, and other low areas. Some dewatering controls, such as the temporary sediment tank, may also be useful in removing other pollutants.

3.3.2 Applications

Dewatering controls are applicable whenever water must be pumped from a low area on a construction site before construction can continue in that area. Pumping of foundations, excavated trenches, and utility vaults are common on development projects.

Dewatering controls may also apply when a temporary cofferdam has been constructed to dewater a normally wet area for construction, such as road crossings of creeks and bank stabilization projects. Water pumped from these areas must be flow through a control before it is discharged back to the water body.

3.3.3 Design Criteria

General

- Construction plan notes shall prohibit the discharge of water from dewatering activities into public streets, flumes, storm drains, creeks or other drainage ways unless controlled to remove suspended soil or other pollutants.
- The designer shall determine whether dewatering will be a batch operation after storm events or a continuous operation due to high groundwater and specify controls accordingly. Controls for continuous dewatering need to provide effective removal of sediment over long periods. Controls that clog easily are not appropriate for controlling long-term dewatering operations.
- Pumped water that has sheen or other evidence of pollutants shall be collected and sampled before it is discharged. State or local discharge permit requirements may exist for the pollutant(s) suspected of being in the water.
- Regulations or effluent criteria that apply to stormwater discharges from a construction activity typically also apply to water discharged from dewatering activities.
- The dewatering controls in this section are most effective with sands and coarse silts. Dewatering controls may be combined with a passive treatment system to provide higher sediment removal rates for fine silt and clay soil particles. Liquid polymers injected at the pump or solid and gel forms installed at the discharge generally work well to promote floc growth and settling of clay soil. Design criteria are contained in *Section 3.7 Passive Treatment System*.

Conventional Controls

- Discharges from dewatering are typically concentrated and have relatively high flow rates and velocities. If conventional controls are used, velocity dissipaters and/or flow spreaders or levelers are required before the control to prevent the discharge from causing erosion and damaging the control.
- The best control for pumped water is to discharge it to a vegetated area.
- Pumped water should be sprayed through a nozzle on the end of a discharge hose or directed to a device that dissipates velocity and disperses flow before the water enters the vegetated area.
- The vegetated area must be large enough to detain the volume being dewatered. The size of area needed is dependent on type of vegetation (interception storage and water uptake capacity) and soil type (infiltration rate) and condition (wet or dry). Vegetation may not be a feasible option if dewatering is due to a large or prolonged storm event and the vegetated area is saturated or if the soil has high clay content.

- If a vegetated area is not available or feasible, the discharge from dewatering may be directed to a conventional sediment barrier, such silt fence, organic filter tubes, sediment basin, or stone outlet sediment trap.
- Opportunities for using the water onsite should be considered, particularly where groundwater intrusion results in frequent or continuous dewatering. The water may be collected in a temporary, onsite storage container or holding pit and used to water vegetation for stabilization, applied for dust control, or used for pavement subgrade preparation. If any of these water needs are present at the time of dewatering, the water may be applied directly to this use without sediment controls, since no discharge occurs.

Sediment Filter Bag

- Sediment filter bags are specifically designed to control pumped water and connect directly to the pump discharge line.
- Show location of the filter bag on the drawings. The bag installed where its discharge will flow away from the disturbed area and onto vegetation or into a swale or drainage ditch with erosion and sediment controls.
- Bags should be placed on a level, stable surface that is prepared with mulch, straw, small aggregate, or other material as recommended by the manufacturer. In some cases, the bag may be placed directly on vegetation or well graded soil. The key is to have a surface without rocks or other protrusions that could puncture the bag.
- The bag should be made of a non-woven, needle-punched, geotextile that meets the following minimum criteria:
  - 205 lbs minimum tensile strength using ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles.
  - 130 lbs minimum puncture strength using ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
  - 400 psi minimum Mullen burst strength using ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method.
  - Minimum 70 percent at 500 hours ultraviolet resistance using ASTM D4355 Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture, and Heat in a Xenon Arc Type Apparatus.
  - 85 to 110 gpm/ft^2 water flow rate using ASTM D4491 Standard Test Methods for Water Permeability of Geotextiles by Permittivity.
- Apparent opening size using ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile should be specified based on the type of soil that will be in the discharge. A size that is too large will not trap the sediment; however, a size that is too small will create an unnecessary head for the dewatering pump to work against.
- The smallest apparent opening size currently available is 70 microns. This size will not capture fine silt and clay particles. A passive treatment system will be necessary with the bag to capture these soils.
- Bags are available in sizes ranging from 6 feet x 6 feet to 15 feet x 25 feet. The size of the bag should be specified based on availability of space, flow rates, and duration of use. If space is available, larger bags will last longer between replacements and may have a lower price per square foot. However, larger bags are heavier when sediment-laden. Equipment must be available to lift and remove the bag from the site for disposal.
- Bags are not reusable. Make sure they are installed at a location where equipment has access to the bags for lifting and removal without causing erosion or damaging other erosion and sediment controls.

Temporary Sediment Tank

- A temporary sediment tank is a compartmented container through which sediment-laden water is pumped to trap and retain sediment before discharging the water to drainage ways, adjoining properties, and rights-of-way below the sediment tank site.
- A temporary sediment tank is typically used at construction sites in urban areas where conventional methods of sediment removal are not practical. It is also used on sites where excavations are deep and space is limited, such as urban construction, where direct discharge of sediment-laden water to streams and storm drainage systems should be avoided.
- The location of temporary sediment tanks should facilitate easy cleanout and disposal of the trapped sediment to minimize interference with construction activities and pedestrian traffic. The tank size should be determined according to the storage volume of the sediment tank, with 1 cubic foot of storage for each gallon per minute of pump discharge capacity.
- A temporary sediment tank can be used as either a sedimentation or filtration device. If an oil sheen is present in the runoff, an underflow baffle may be used in the tank to remove it. However, local and state discharge regulations and permits may apply and should be checked before discharging.
- For use as a small scale sedimentation basin, de-watering discharge is directed into the temporary sediment tank to a level below the tank midpoint and held for a minimum of 2 hours to allow settlement of a majority of the suspended particles. This detention time is insufficient for removal of fine silt and clay soil particles. Passive treatment systems should be combined with the tank if these soil particles will be present.
- The tank should be designed for a controlled release when the contents of the tank reach a level higher than the midpoint.
- As a filtration device, a temporary sediment tank is used for collecting de-watering discharge and passing it through a filtered opening at the outlet of the tank to reduce suspended sediment volume. The filter opening in the temporary sediment tank should have an Apparent Opening Size (AOS) (see *Section 3.10 Silt Fence*) of 70 or smaller.
- The trapped sediment and stormwater must be disposed of properly.

3.3.4 Design Guidance and Specifications

No specification for dewatering controls is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.3.5 Inspection and Maintenance Requirements

Dewatering controls should be inspected regularly (at least as often as required by the TPDES Construction Permit). Dewatering discharge points should be checked for erosion. Eroded areas should be repaired, and erosion controls should be installed to prevent future erosion.

Dewatering pumps and sediment controls should be monitored frequently, at least hourly, while pumps are in operation to prevent unauthorized discharges and to catch erosion problems or control failure.

Conventional sediment controls should be inspected at least weekly when used for continuous dewatering, because they will become overcome with sediment more quickly than when used to control runoff from storm events. The controls shall be maintained according to the criteria in their respective sections. They should be replaced when they no longer provide the necessary level of sediment removal.

Sediment filter bags should be checked to determine if they need replacing. The bags cannot be cleaned or reused. They should be used until they reach the manufacturer's recommended capacity. The entire bag with sediment can be disposed of as solid waste. If a controlled location onsite or a spoil site is available, the bag can be cut open and the sediment spread on the ground. Only the bag is waste in this case.

Sediment tanks should be cleaned when they become ⅓ full of sediment. To facilitate maintenance, the tanks need to be located with easy access for regular pump out. The rate at which a tank is pumped depends on site-specific considerations such as rainfall and sediment loads to the system. Regular inspections will help determine pump out frequency and prevent overloading and failure of the system.

3.3.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

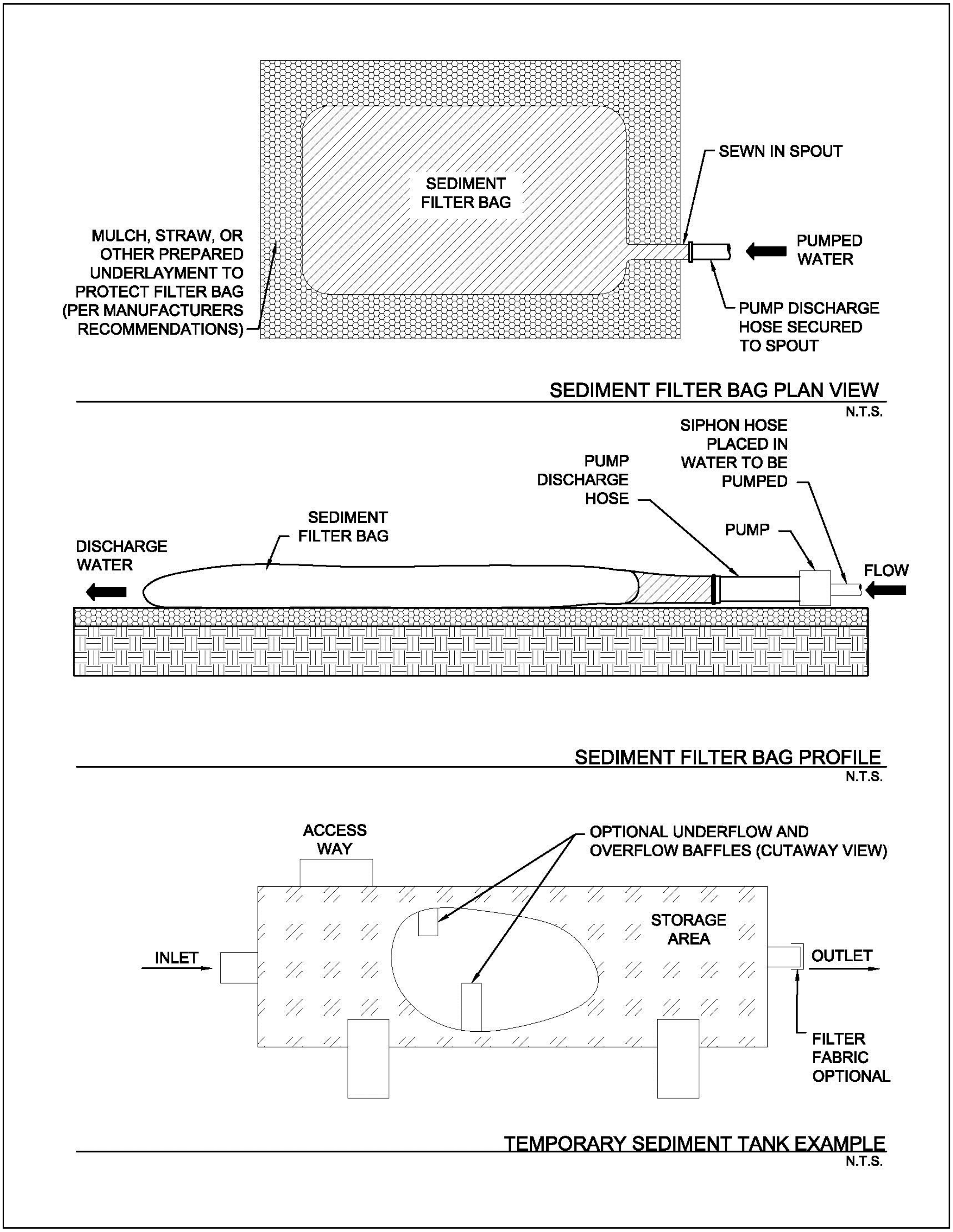
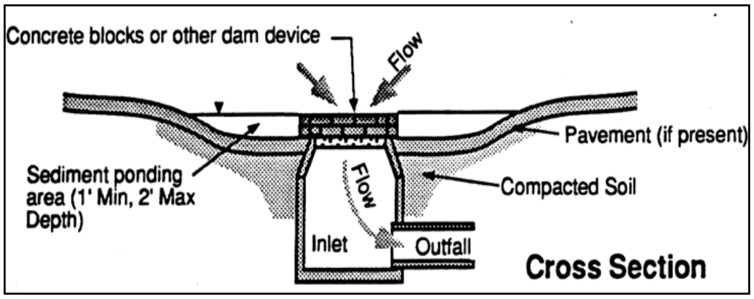


Figure 3.4 Schematics of Dewatering Controls

3.4 Inlet Protection

Sediment Control



Description: Inlet protection consists of a variety of methods to intercept sediment at low point inlets through the use of depressed grading, filter stone, filter fabric, inlet inserts, organic filter tubes and other materials. The protection devices are placed around or across the inlet openings to provide localized detention or filtration of sediment and floatable materials in stormwater. Protection devices may be assembled onsite or purchased as manufactured assemblies.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Evaluate drainage patterns to ensure inlet protection will not cause flooding of roadway, property or structures
- Never block entire inlet opening
- Size according to drainage area and flow rates
- Include flow bypass for clogged controls and large storm events

ADVANTAGES / BENEFITS:

- May be the only feasible sediment control when all construction is located within rights-of-way

DISADVANTAGES / LIMITATIONS:

- Limited effectiveness and reliability
- High maintenance requirements
- Has potential to flood roadways or adjacent properties

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Check for and remove blockage of inlet after every storm event
- Remove sediment before it reaches half the design height or volume of the inlet protection, more frequently for curb inlets
- Repair or replace damaged materials
- Clean or replace filter stone and organic filter tubes is when clogged with sediment

TARGETED POLLUTANTS

◒ Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
◒ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.35-0.65
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
● Maintenance
○ Training
○ Suitability for Slopes > 5%

Other Considerations:

- *Traffic hazards*
- *Passage of larger storm events without causing flooding*
- *Flow diversion to other inlets or drainage points*

3.4.1 Primary Use

Inlet protection is typically used as a secondary sediment barrier, due to its limited effectiveness and numerous disadvantages. It is used to reduce sediment in storm sewer systems by serving as a back-up system for areas that have newly applied erosion controls or for other sediment controls that cannot achieve adequate sediment removal by themselves.

Inlet protection may be used as a primary sediment control only when all other primary controls are infeasible because of site configuration or the type of construction activity.

3.4.2 Applications

Inlet protection is best applied at low point (sump) inlets where stormwater runoff will pond behind the protection measure, and then either filter through the protection measure or flow over a weir created by it. Most inlet protection measures depend on ponding to be effective. These types of inlet protection are not applicable to on-grade curb inlets, where the inlet protection will cause stormwater runoff to bypass the inlet and overload downstream inlets. Only inlet protection measures that allow for use of the inlet opening (e.g. inlet inserts) are applicable as inlet protection for on-grade inlets.

Inlet protection is normally used in new developments with new inlets and roads that are not in public use. It has limited applications in developed areas due to the potential for flooding, traffic safety, pedestrian safety, and maintenance problems. Potential applications in developed areas are on parking lot inlets where water can pond without causing damage and during major repairs to existing roadways where no other controls are viable.

The application of inlet protection is highly variable due to the wide variety of inlet configurations (existing and new) and site conditions. The schematics in Section 6 show example applications; however, applications in most cases must be site adapted. Different methods and materials may be used. It is the responsibility of the designer to ensure that the methods and materials applied for inlet protection are appropriate to the site and flow conditions following the design criteria in Section 3.

3.4.3 Design Criteria

General

- Drainage patterns shall be evaluated to ensure inlet protection will not divert flow or flood the roadway or adjacent properties and structures.
- Inlet protection measures or devices that completed block the inlet are prohibited. They must also include a bypass capability in case the protection measures are clogged.
- Inlet protection must be designed to pass the conveyance storm (25-year, 24-hour) without creating a road hazard or damaging adjacent property. This may be accomplished by any of the following measures:
  - An overflow weir on the protection measure.
  - An existing positive overflow swale on the inlet.
  - Sufficient storage volume around the inlet to hold the ponded water until it can all filter into the inlet.
  - Other engineered method.
- Positive overflow drainage is critical in the design of inlet protection. If overflow is not provided for at the inlet, temporary means shall be provided to route excess flows through established swales, streets, or other watercourses to minimize damage due to flooding.
- Filter fabric and wire mesh used for inlet protection shall meet the material requirements specified in *Section 3.10 Silt Fence*.

- Block and gravel (crushed stone or recycled concrete) protection is used when flows exceed 0.5 cubic feet per second and it is necessary to allow for overtopping to prevent flooding.
- The tube and filler for organic filter tubes shall be in accordance with the criteria in *Section 3.6 Organic Filter Tube*.
- Bags used to secure inlet protection devices on pavement shall be filled with aggregate, filter stone, or crushed rock that is less likely than sand to be washed into an inlet if the bag is broken. Filled bags shall be 24 to 30 inches long, 16 to 18 inches wide, and 6 to 8 inches thick. Bags shall be polypropylene, polyethylene, or polyamide woven fabric with a minimum unit weight of 4 ounces per square yard and meet the following criteria:
  - Greater than 300 psi Mullen Burst Strength using ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method.
  - Greater than 70 percent UV Stability using ASTM D4355 Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture, and Heat in a Xenon Arc Type Apparatus.

Curb Inlet Protection

- Municipality approval is required before installing inlet protection on public streets.
- Special caution must be exercised when installing curb inlet protection on publicly traveled streets or in developed areas. Ensure that inlet protection is properly designed, installed and maintained to avoid flooding of the roadway or adjacent properties and structures.
- A two inch overflow gap or weir is required on all curb inlet protection devices.
- Traffic cones, warning signs, or other measures shall be installed to warn motorists when the inlet protection measures extend beyond the gutter line.
- 2 inch X 4 inch Weir Protection:
  - Bend wire mesh around the 2 inch x 4 inch board and staple to the board. Bend wire mesh around the bottom of the board, the curb opening, and along the pavement to form a cage for the rock.
  - Rock bags shall be placed perpendicular to the curb, at both ends of the wooden frame, to disrupt the flow and direct water into the rock filter. Stack the bags two high if needed.
- Organic Filter Tube Protection:
  - The diameter of the tube shall be at least 2 inches less than the height of the inlet opening. The tube should not be allowed to block the entire opening, since it will clog.
  - The tube shall be placed on 4 inch x 4 inch or 2 inch x 4 inch wire mesh to prevent the tube from sagging into the inlet.
  - The tube should be long enough to extend a minimum of 12 inches past the curb opening on each side of the inlet.
- Hog Wire Weir Protection:
  - The filter fabric and wire mesh shall extend a minimum of 12 inches past the curb opening on each side of the inlet.
  - Filter fabric shall be placed on 2 inch x 4 inch wire mesh to prevent the tube from sagging into the inlet.
  - Rock bags are used to hold the wire mesh and filter fabric in contact with the pavement. At least one bag shall be placed on either side of the opening, parallel to and up against the concrete curb. The bags are in intended to disrupt and slow the flow and ensure it does not go under the fabric. Add bags if needed.

  - If a board is used to anchor the wire mesh and fabric instead of rock bags, the board shall be secured with concrete nails at 3 inches on center. Upon removal clean any dirt or debris from the nailing locations, apply chemical sanding agent, and apply non-shrink grout flush with surface of concrete.

- Block and Gravel Protection:
  - Concrete blocks shall be standard 8 inch x 8 inch x 16 inch concrete masonry units and shall be in accordance with ASTM C139, Concrete Masonry Units for Construction. Filter gravel shall be ¾ inch washed stone containing no fines. Angular shaped stone is preferable to rounded shapes.
  - Concrete blocks are to be placed on their sides in a single row around the perimeter of the inlet, with ends abutting. Openings in the blocks should face outward, not upward. ½ inch x ½ inch wire mesh shall then be placed over the outside face of the blocks covering the holes. Filter gravel shall then be piled against the wire mesh to the top of the blocks with the base of the stone being a minimum of 18 inches from the blocks.
  - Alternatively, where loose stone is a concern (streets, etc.), the filter gravel may be placed in appropriately sized filter fabric bags.
  - Periodically, when the gravel filter becomes clogged, the gravel must be removed and cleaned in a proper manner or replaced with new gravel and piled back against the wire mesh.
- Organic Filter Tube On-Grade Protection:
  - Organic filter tubes may be used to provide sediment control at on-grade curb inlets where the tube will not be a traffic hazard, such as on residential streets where the pavement adjacent to the curb is allocated to parked cars. Tubes should not be used in this manner where they will extend into an active travel lane.
  - The filter tube shall be secured in a U-shape by rock bags. Runoff flowing in the gutter will pond within the U until it filters through the tube or overflows around the end.
- Inlet protection shall be phased on curb inlets being constructed. Controls shall be installed on the pipe inlet at the bottom of the catch basin as soon as it is installed and while the inlet box and top are being formed or placed.

Area Inlet Protection

- Installation methods for protection on area inlets vary depending on the type of inlet (drop, "Y," or other) and the type and use of the surface surrounding the inlet (parking lot, playground, etc.). It is the responsibility of the designer to appropriately adapt inlet protection measures and their installation methods for each site condition. Several types may be needed on one project.
- Filter Fabric Protection:
  - Filter fabric protection is appropriate where the drainage area is less than one acre and the basin slope is less than five (5) percent. Filter fabric, posts, and wire mesh shall meet the material requirements specified in *Section 3.10 Silt Fence*.
  - A 6 inch wide trench is to be cut 6 inches deep at the toe of the fence to allow the fabric to be laid below the surface and backfilled with compacted earth or gravel. This entrenchment prevents any bypass of runoff under the fence.
  - Stone overflow structures, according to the criteria in *Section 3.10 Silt Fence* shall be installed where flow to the inlet is concentrated and more than 1 cubic feet per second.
- Excavated Impoundment Protection:
  - Excavated inlet protection is usually the most effective type of area inlet protection; however, it is only applicable to drop inlets. It should not be applied to Y inlets because it will undermine the concrete pad surrounding the inlet opening. Nor can it be used for inlets on pavement.

  - With this protection method, it is necessary to install weep holes to allow the impoundment to drain completely.
  - The impoundment shall be sized such that the volume of excavation is equal to or exceeds the runoff volume from the temporary control design storm (2-year, 24-hour) for the inlet's drainage area.
  - The trap shall have a minimum depth of one foot and a maximum depth of 2 feet as measured from the top of the inlet and shall have side slopes of 2:1 or flatter.
- Block and Gravel Protection:
  - Block and gravel inlet protection is the most stable area inlet protection and can handle more concentrated flows. It may be installed on paved or vegetated surfaces. Loose stone shall be carefully removed from vegetated surfaces at the end of construction to prevent the stone from becoming a mowing hazard.
  - The inlet protection may be one or two blocks high. Single block heights are applicable for drainage areas up to 3 acres in size. The double block height shall be used for larger drainage areas.
  - Concrete blocks shall be standard 8 inch x 8 inch x 16 inch concrete masonry units and shall be in accordance with ASTM C139, Concrete Masonry Units for Construction. Filter gravel shall be ¾ inch washed stone containing no fines. Angular shaped stone is preferable to rounded shapes.
- Organic Filter Tube Protection:
  - Organic filter tubes may be used on paved or unpaved surfaces.
  - On paved surfaces, tubes shall be secured in place by rock bags. On unpaved surfaces, the tubes shall be embedded in the ground a minimum of 3 inches and staked at 4 foot spacing.
  - Designer shall provide calculations and specify the diameter of tube to be used based on the inlet's drainage area and the flow rate of runoff to the inlet. The minimum allowable diameter is 12 inches.

Proprietary Inlet Protection

- Numerous proprietary protection devices are available from commercial vendors. The devices often have the advantage of being reusable on several projects if they are maintained in good condition.
- It is the policy of this manual not to recommend any specific commercial vendors for proprietary controls. However, this subsection is included in order to provide municipalities with a rationale for approving the use of a proprietary inlet protection device within their jurisdiction.
- The designer shall work with the supplier to provide the municipality with flow calculations or independent third-party tests that document the device's performance for conditions similar to the ones in which it is proposed to be installed. The conditions that should be considered include: type and size of inlet, inlet configuration, size of contributing drainage area, design flow rate, soil particle sizes to be removed, and other pollutants to be removed.
- The designer or vendor of the proprietary device shall provide a minimum of three references for projects where the device has been installed and maintained in operation at a construction site for at least six months. Local references are preferred; but references from other regions can be accepted if a similarity between the reference project and the proposed application can be demonstrated.
- Proprietary devices must not completely block the inlet. The device shall have a minimum of a 2 inch wide opening for the length of the inlet when it will be used in areas that water can safely pond to depths deeper than the design depths for the inlet. If ponding is not an option, then the device must have overflow capacity equal to the inlet design flow rate.
- Some proprietary devices are available with replaceable pads or filters. These pads or filters have the added benefit or removing pollutants such as metals and oils in addition to removing sediment.

These types of inserts are recommended in applications where prior or current land use in or adjacent to the construction areas may result in the discharge of pollutants.

- Proprietary protection devices shall be in accordance with the General criteria at the beginning of this section and any criteria listed under Curb Inlet Protection and Area Inlet Protection that are not specific to an inlet protection method.

3.4.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.15 Inlet Protection.

3.4.5 Inspection and Maintenance Requirements

Inlet protection should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Inlet controls should also be inspected after every storm event to check for collapse into the inlet or other damages that may block flow in the inlet. In addition to routine inspection, inlet protection devices should be observed and monitored during larger storm events to verify that they are not ponding or diverting water in a manner that floods a roadway or damages property.

Floatable debris and other trash caught by the inlet protection should be removed after each storm event. Sediment should also be removed from curb inlet protection after each storm event because of the limited storage area associated with curb inlets.

Sediment collected at area inlet protection should be removed before it reaches half the height of the protection device. Sediment should be removed from inlets with excavated impoundment protection before the volume of the excavation is reduced by 50 percent. In addition, the weep holes should be checked and kept clear of blockage.

Concrete blocks, 2 inch x 4 inch boards, stakes, and other materials used to construct inlet protection should be checked for damaged and repaired or replaced if damaged.

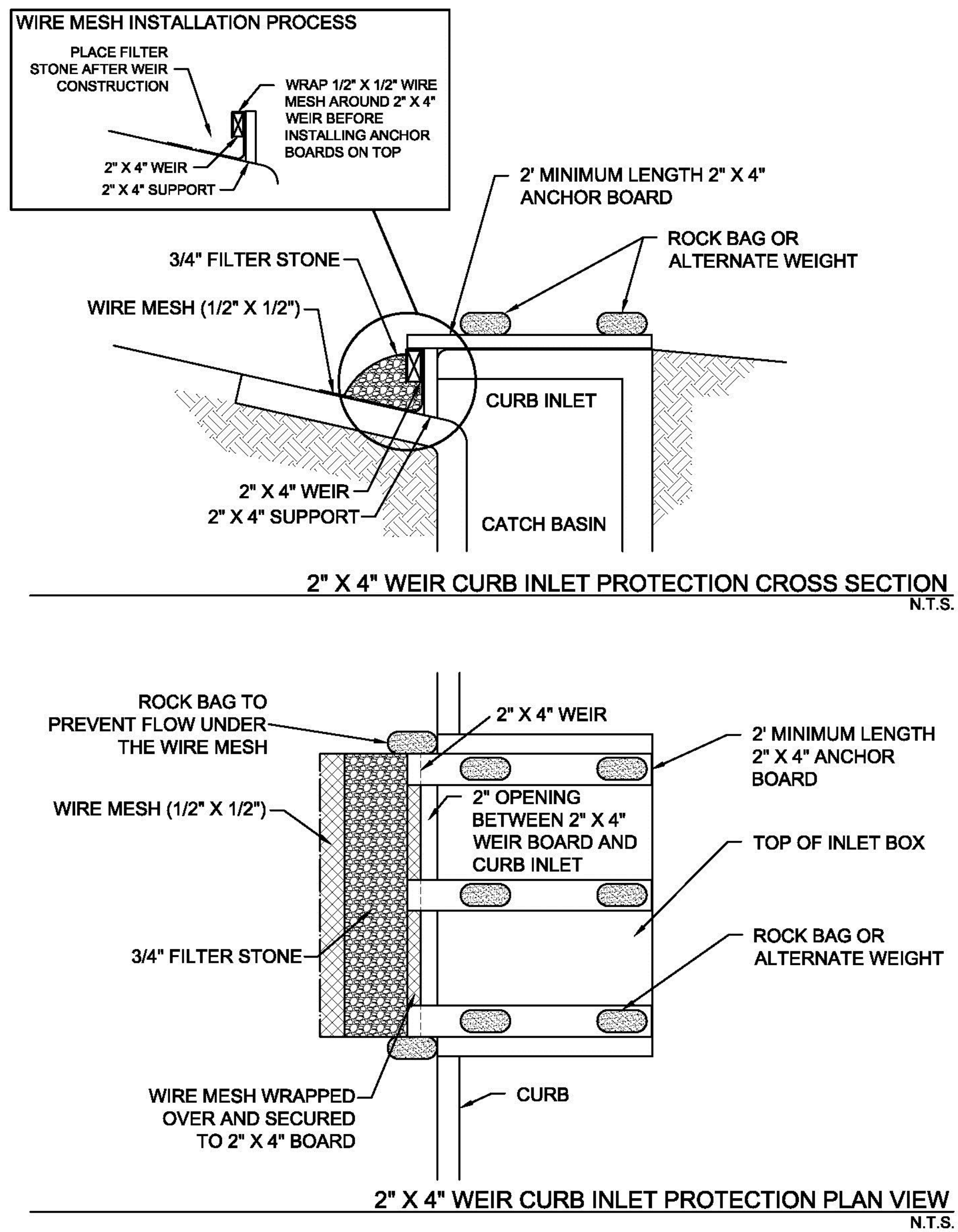
When filter fabric or organic filter tubes are used, they should be cleaned or replaced when the material becomes clogged. For systems using filter stone, when the filter stone becomes clogged with sediment, the stones must be pulled away from the inlet and cleaned or replaced.

Because of the potential for inlet protection to divert runoff or cause localized flooding, remove inlet protection as soon as the drainage area contributing runoff to the inlet is stabilized. Ensure that all inlet protection devices are removed at the end of the construction.

3.4.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



NOTE: THIS CONTROL WILL DECREASE THE CAPACITY OF THE INLET. IT SHALL ONLY BE USED WHEN AN ENGINEER HAS DETERMINED THERE IS ADEQUATE STORAGE OR POSITIVE OVERFLOW.

Figure 3.5 Schematics of 2”x4” Weir Curb Inlet Protection
(Source: Modified from Washington Suburban Sanitary Commission Detail SC-16.0)

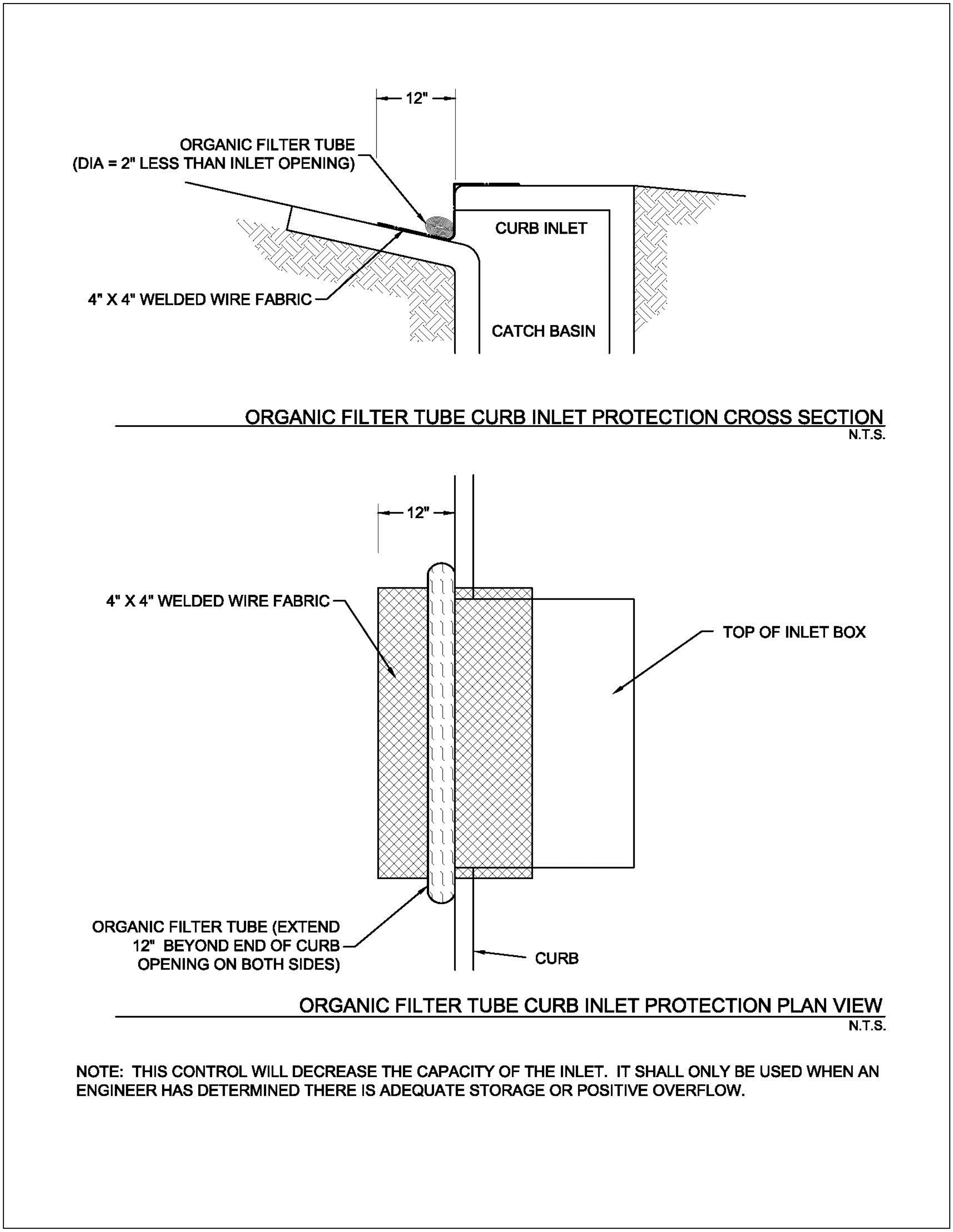


Figure 3.6 Schematics of Organic Filter Tube Curb Inlet Protection
(Source: Modified from City of Plano BMP SP-4)

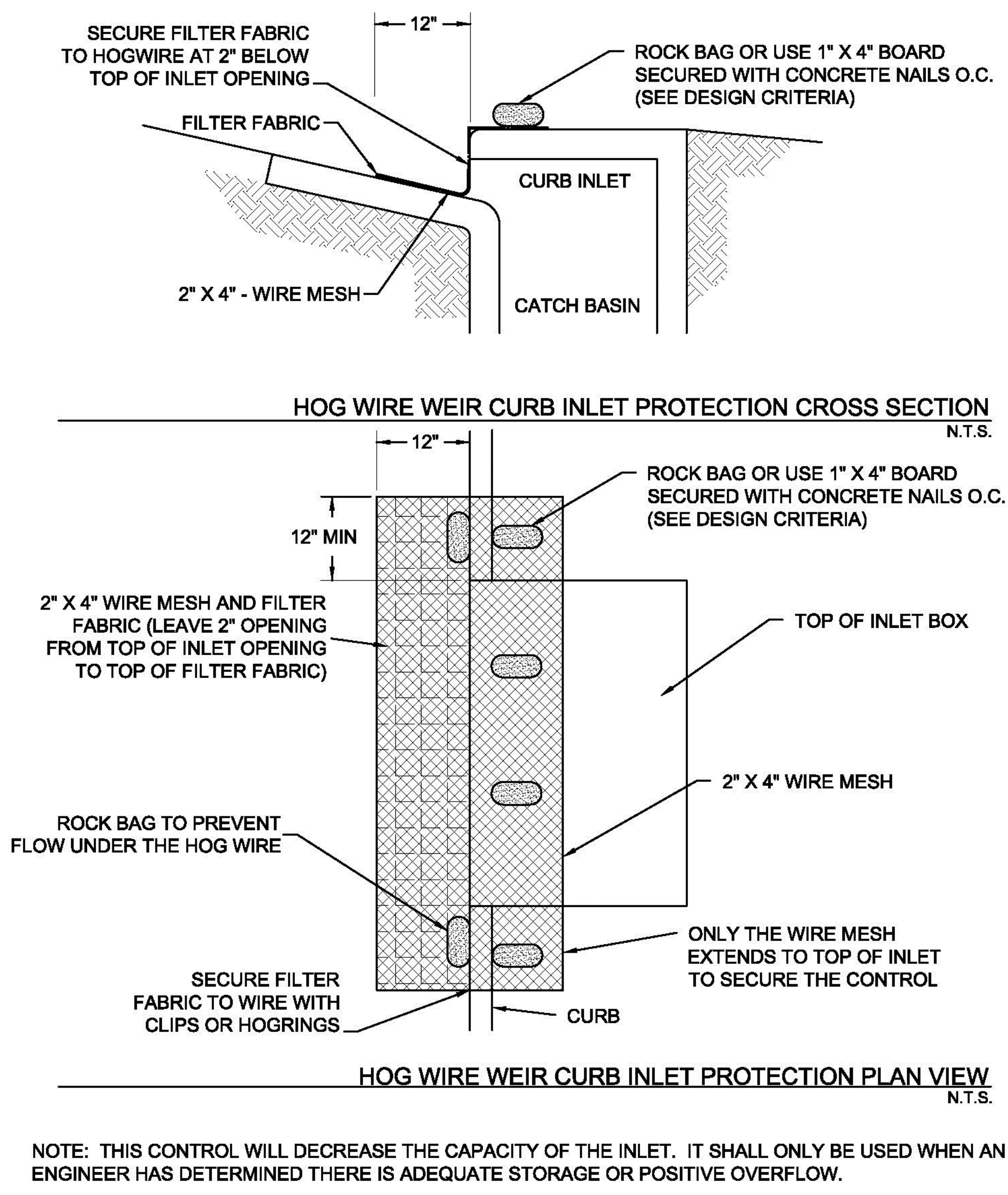
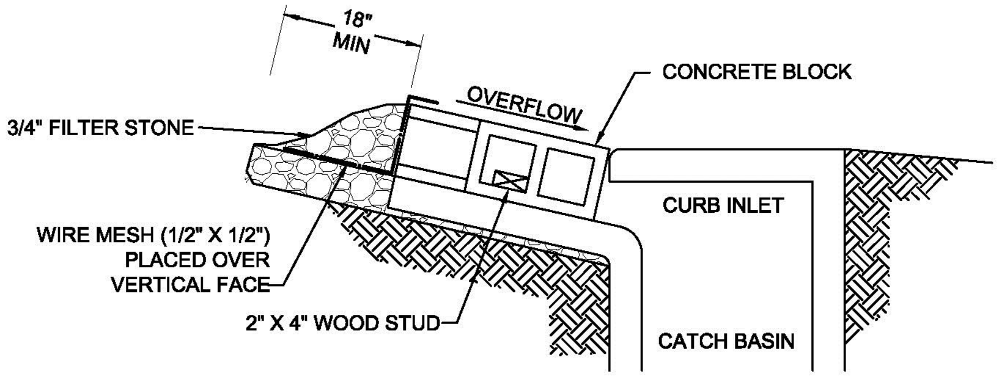
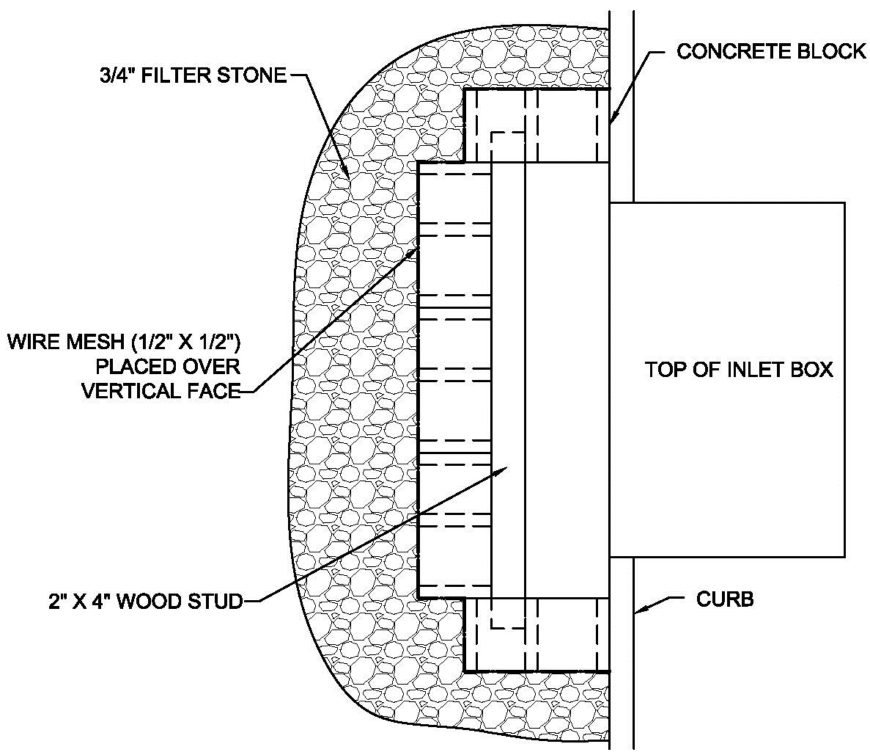


Figure 3.7 Schematics of Hog Wire Weir Curb Inlet Protection
(Source: Modified from City of Round Rock Detail E-03)



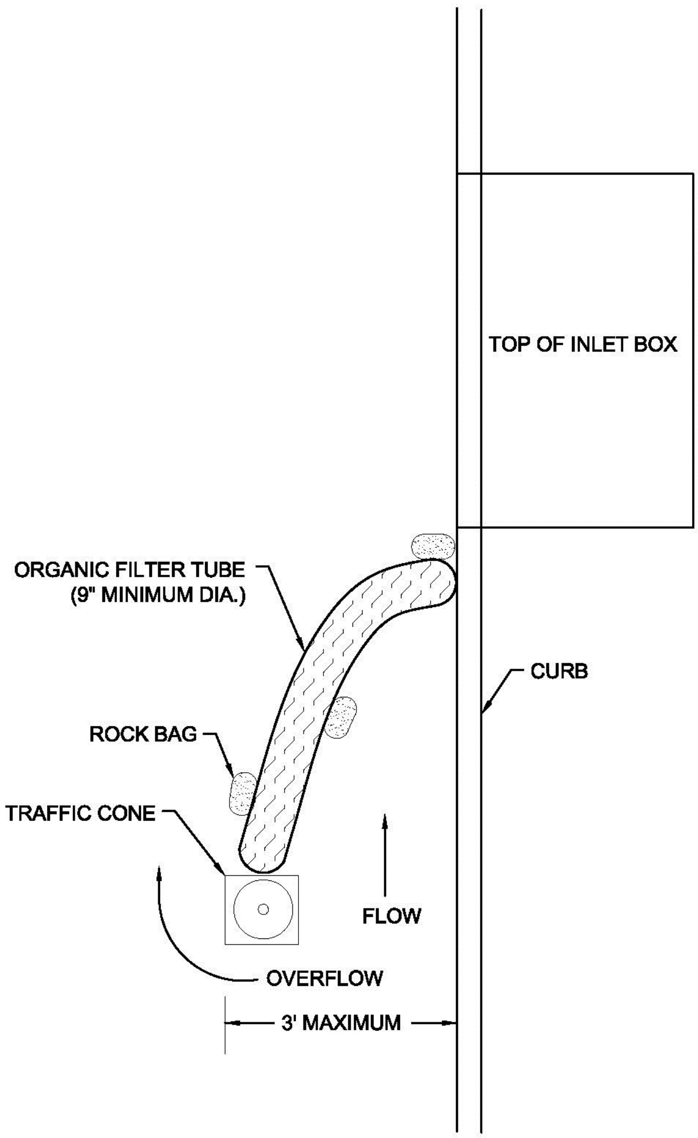
BLOCK AND GRAVEL FILTER CURB INLET PROTECTION CROSS SECTION
N.T.S.



BLOCK AND GRAVEL FILTER CURB INLET PROTECTION PLAN VIEW
N.T.S.

NOTES:
1. DO NOT INSTALL ON INLETS IN A PUBLIC STREET OR OTHER ACTIVE TRAVEL LANE. BLOCK AND GRAVEL FILTER IS INTENDED FOR USE ON LOW POINT (SUMP) INLETS IN PARKING LOTS AND OTHER PAVEMENT THAT IS NOT AN ACTIVE TRAVEL LANE. THIS INLET PROTECTION METHOD ALLOWS FOR FULL USE OF THE INLET DESIGN CAPACITY.
2. INSTALL TRAFFIC CONES AS NEEDED TO MINIMIZE THE POTENTIAL FOR CARS HITTING THE BLOCK AND GRAVEL.

Figure 3.8 Schematics of Block and Gravel Filter Curb Inlet Protection

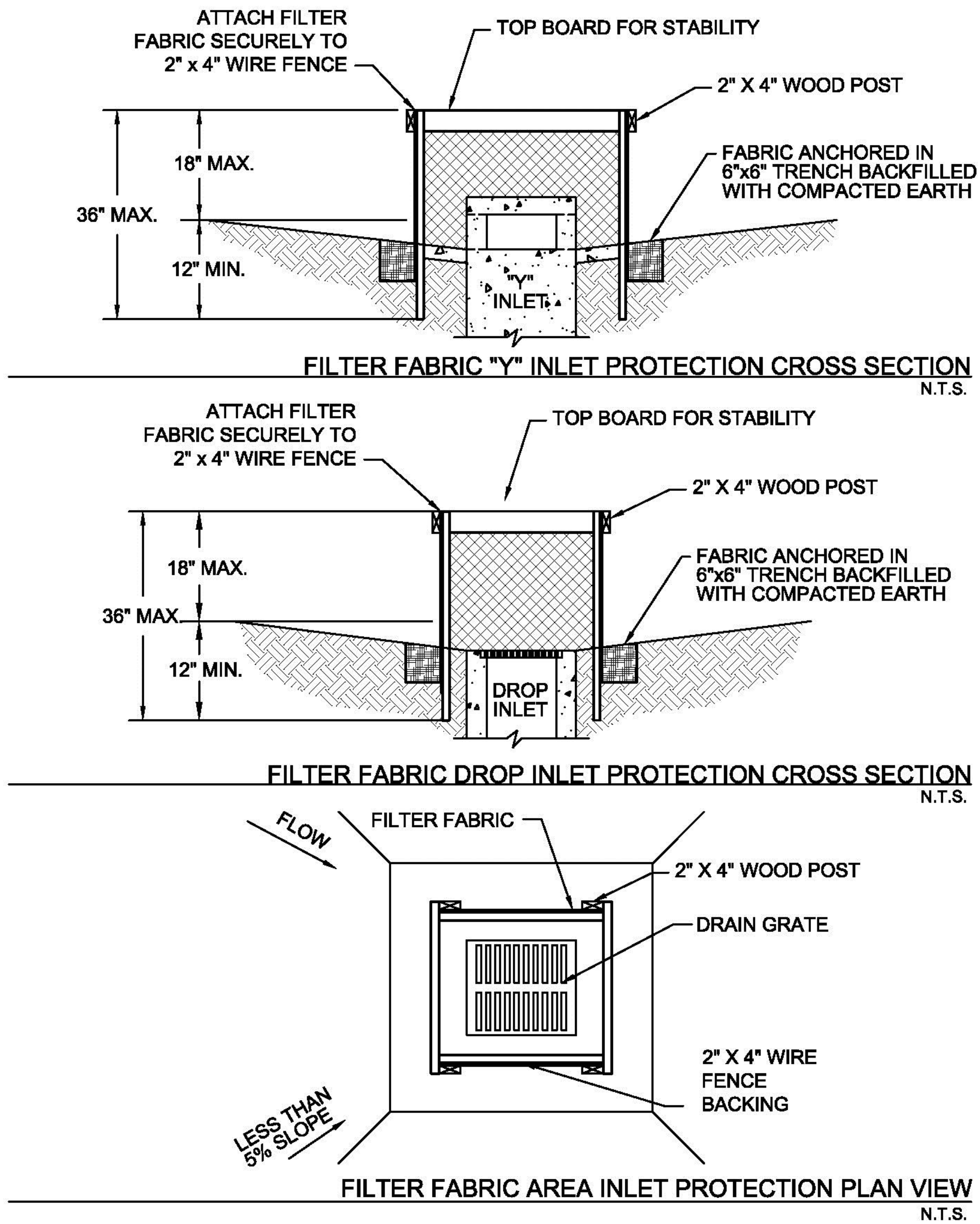


ORGANIC FILTER TUBE ON-GRADE CURB INLET PROTECTION DETAIL

N.T.S.

NOTES:
1. THIS DETAIL IS INTENDED FOR USE WITH ON-GRADE INLETS (NOT A LOW POINT) WHERE WATER WOULD BE DIVERTED INSTEAD OF PONDING BEHIND THE OTHER TYPES OF INLET PROTECTION.
2. DO NOT INSTALL ON INLETS WHERE THE ORGANIC FILTER TUBE WOULD EXTEND INTO AN ACTIVE TRAVEL LANE.

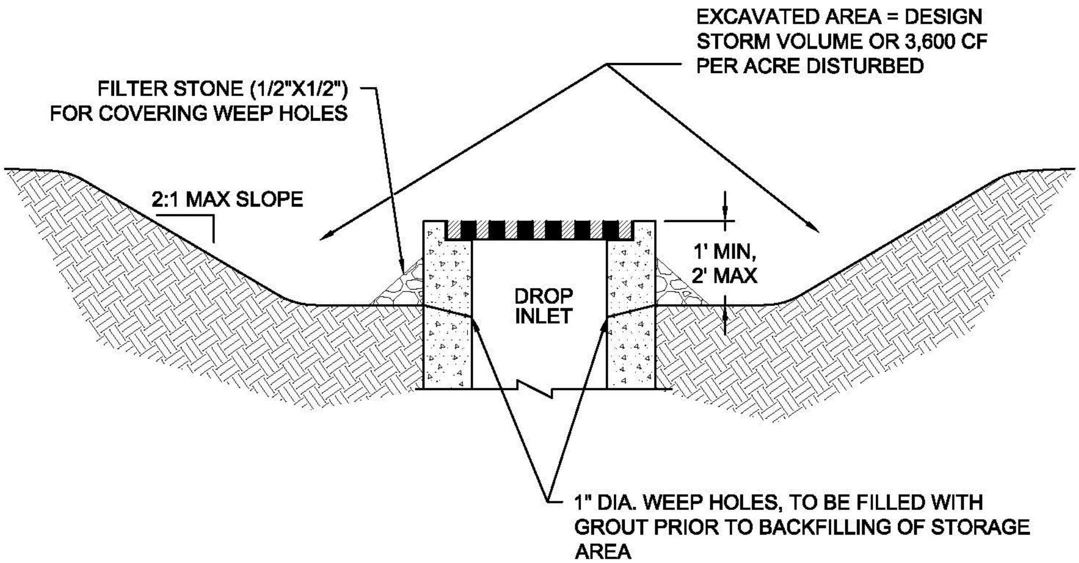
Figure 3.9 Schematic of Organic Filter Tube On-Grade Curb Inlet Protection



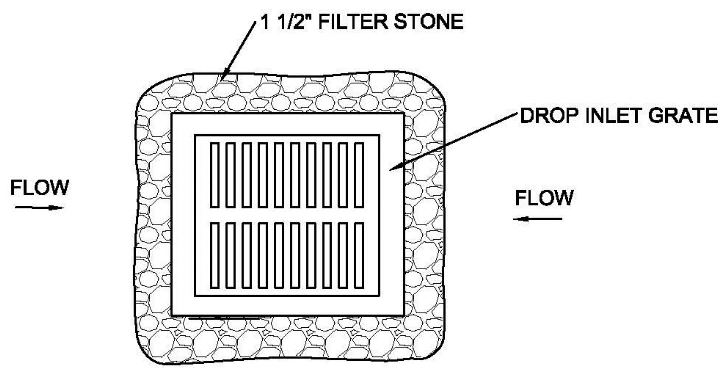
NOTES:
1. CONCENTRATED DITCH FLOW COMING FROM ONE OR MORE SIDES TOWARD THE INLET MAY REQUIRE A STONE OVERFLOW STRUCTURE TO BE CONSTRUCTED ON ONE SIDE OF THE INLET (SEE *SECTION 3.10 SILT FENCE*).
2. POSTS SHALL BE INSTALLED AT EACH CORNER AND BETWEEN CORNERS IF THE DISTANCE IS GREATER THAN 6' BETWEEN CORNER POSTS.

Figure 3.10 Schematics of Filter Fabric Area Inlet Protection

(Source: City of Plano BMP SP-4)

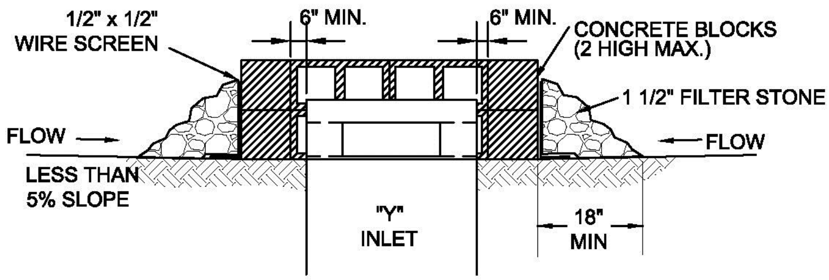


EXCAVATED IMPOUNDMENT DROP INLET PROTECTION CROSS SECTION
N.T.S.

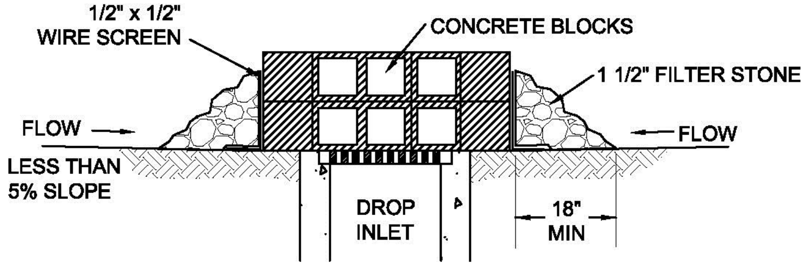


EXCAVATED IMPOUNDMENT AREA INLET PROTECTION PLAN VIEW
N.T.S.

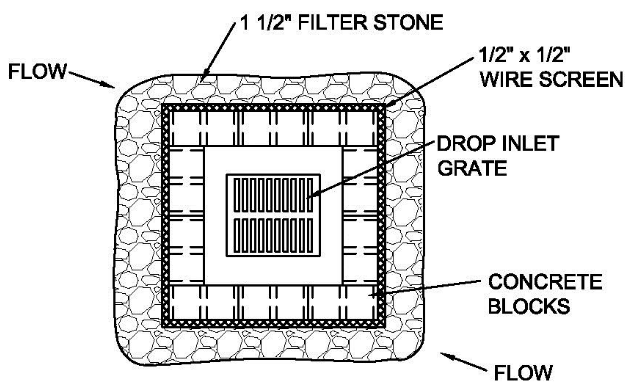
Figure 3.11 Schematics of Excavated Impoundment Area Inlet Protection



BLOCK AND GRAVEL "Y" INLET PROTECTION CROSS SECTION
N.T.S.



BLOCK AND GRAVEL DROP INLET PROTECTION CROSS SECTION
N.T.S.



BLOCK AND GRAVEL AREA INLET PROTECTION PLAN VIEW
N.T.S.

Figure 3.12 Schematics of Block and Gravel Area Inlet Protection
(Source: Modified from City of Plano BMP SP-4)

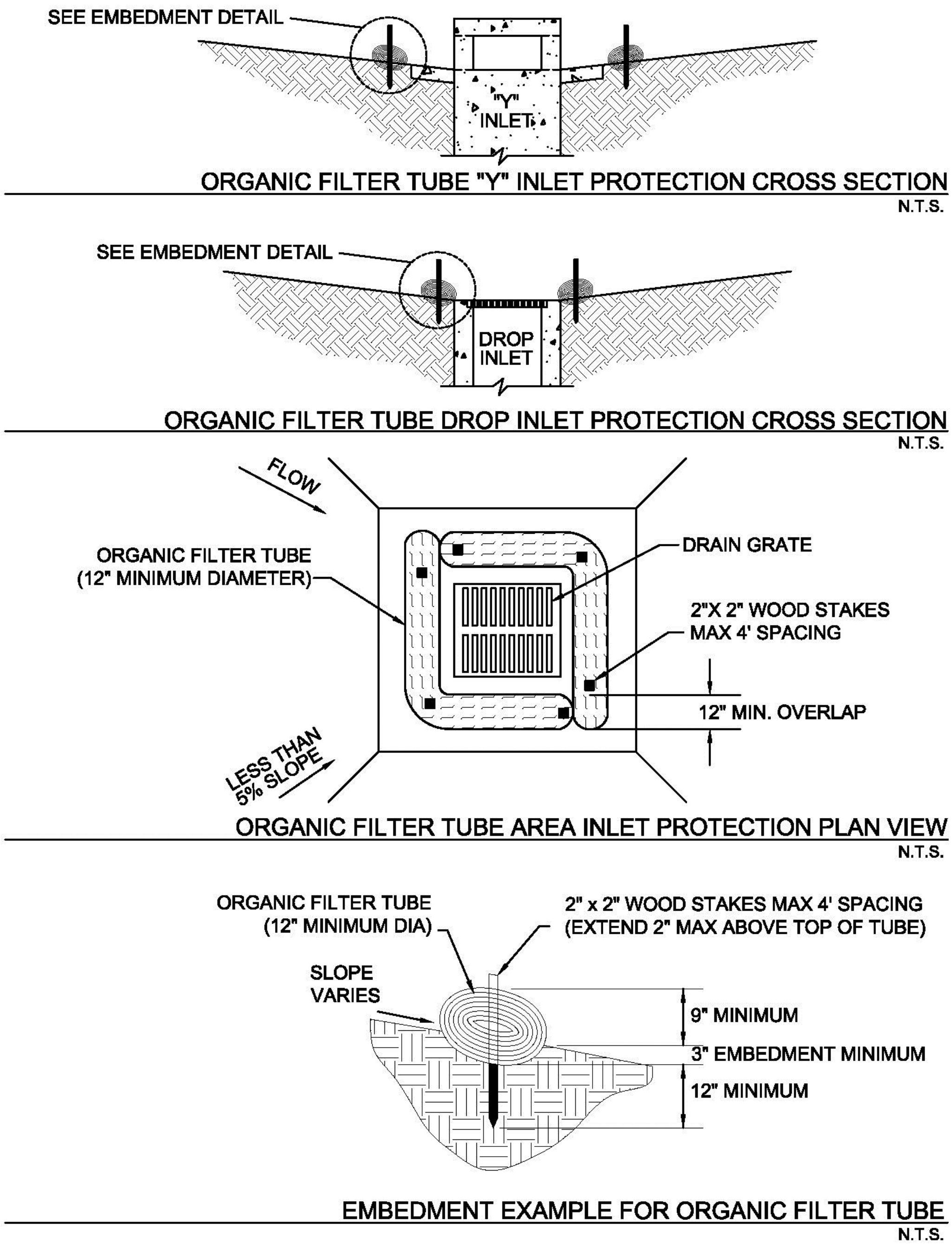
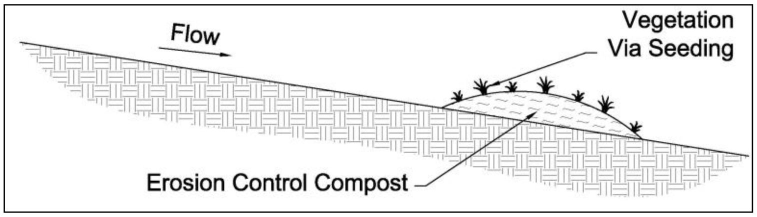


Figure 3.13 Schematics of Organic Filter Tube Area Inlet Protection

3.5 Organic Filter Berm

Sediment Control



Description: Organic filter berms, also called compost filter berms, are linear berms constructed of a mix of compost and wood chips. They are placed on a contour to control runoff. The organic filter berm provides both filtration and time for sediment settling by reducing the velocity of the runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum drainage area of 0.25 acre per 100 linear feet of berm
- Maximum 200 feet distance of flow to silt fence; 50 feet if slope exceeds 10 percent
- 1½ to 3 feet high, top width of 2 to 3 feet, and base of 3 to 5 feet for trapezoidal shaped berms
- 1 to 2 feet high and 2 to 4 feet wide for windrow (triangular) berms

ADVANTAGES / BENEFITS:

- Economical means to trap sediment
- Most effective with coarse to silty soil types
- May be tilled into the soil at end of project, thus adding organic content to the soil

DISADVANTAGES / LIMITATIONS:

- Localized flooding due to minor ponding upslope of the filter berm
- Not for use in swales or low areas where berms will be subject to concentrated flow
- Can interfere with construction operations
- Repeated clogging may require replacement of berm with another control

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Repair undercutting and other failures
- Remove sediment when before it reaches one-half the height of the berm
- Maintain dimensions of the berm by replacing organic filter material when necessary

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ◒ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier

Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.50-0.75
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ● Maintenance
- ○ Training
- ◒ Suitability for Slopes > 5%

Other Considerations

- *Effects of ponding on adjacent areas and property*

3.5.1 Primary Use

Organic filter berms are used as perimeter controls down slope of disturbed areas and on side slopes where stormwater may runoff the area. They are very well suited to sites with small disturbed drainage areas that are not subjected to concentrated flows and that will ultimately be seeded, sodded, or landscaped.

3.5.2 Applications

Properly designed, the organic filter berm is economical due to the ease of installation and because it can be tilled into the soil at the end of project, limiting the cost of removal and adding to the organic content of the soil. The berms are used as perimeter control devices for both development sites and linear (roadway) type projects. They are most effective with coarse to silty soil types. Additional controls, such as a passive treatment system, may be needed to remove fine silts and clay soils suspended in stormwater.

3.5.3 Design Criteria

- Filter berms are to be constructed along a line of constant elevation (along a contour line) where possible.
- Berms can interfere with construction operations; therefore planning of access routes onto the site is critical.
- Maximum drainage area shall be 0.25 acre per 100 linear feet of filter berm.
- Maximum flow to any 20 foot section of filter berm shall be 1cubic feet per second.
- Maximum distance of flow to berm shall be 200 feet or less. If the slope exceeds 10 percent the flow distance shall be less than 50 feet.
- Maximum slope adjacent to the filter berm shall be 4:1.
- Trapezoidal shaped berms should be 1½ to 3 feet high with a top width of 2 to 3 feet and a base of 3 to 6 feet wide.
- Windrow (triangular) shaped berms should be 1 to 2 feet high and 2 to 4 feet wide.
- Berm side slopes shall be 2:1 or flatter.
- Roughen the soil surface before placing the berm to increase adherence of the compost.
- Compost shall conform to the requirements for Erosion Control Compost in Item 161 of the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT 2004).
- Organic filter berms should be stabilized by seeding if there are no other sediment controls down slope of the filter berm. Seeding shall be as specified in *Section 2.9 Vegetation* at a seed loading of 1 lb. per 10 linear feet for small berms (1ft. by 2 ft.) or 2.25 lbs per 10 linear ft. for larger berms (1.5 ft. by 3 ft.)

3.5.4 Design Guidance and Specifications

Specifications for Erosion Control Compost to be used as filter material may be found in Item 161 of the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT 2004).

3.5.5 Inspection and Maintenance Requirements

Filter berms should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for buildup of excess sediment, undercutting, and other failures. Silt must be removed when before it reaches half the height of the berm. Silt may be raked from the disturbed side of the device to clean side the berm for the first few times that it becomes clogged to prevent ponding. Repeated clogging of the berm at one location will require replacement of the organic filter material or may require installation of another control to prevent failure of the berm.

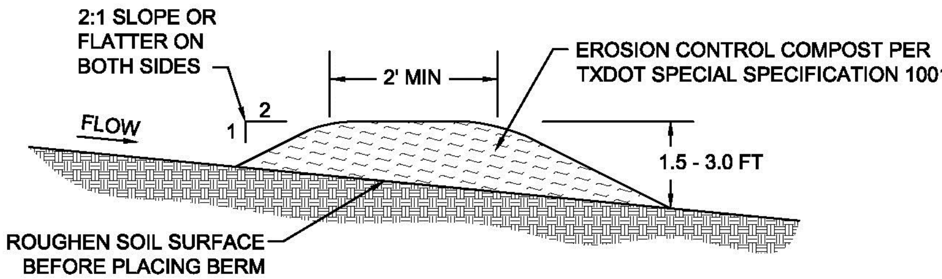
Dimensions of the berm must be maintained by replacing organic filter material when necessary. Typically excess material is stockpiled onsite for repairs to berms disturbed by construction activity.

There shall be no signs of erosion, breeching or runoff around or under the berm.

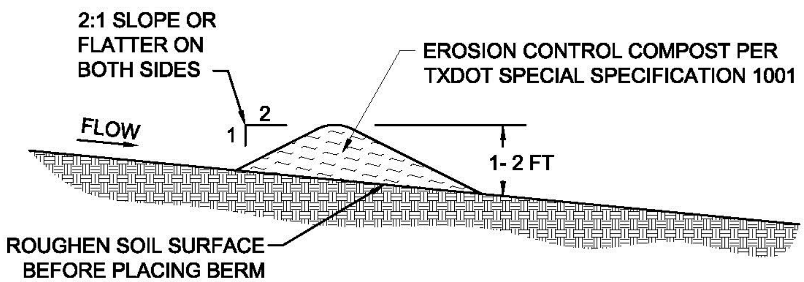
3.5.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



TRAPEZOIDAL SHAPED ORGANIC FILTER BERM
N.T.S.



TRIANGULAR SHAPED ORGANIC FILTER BERM
N.T.S.

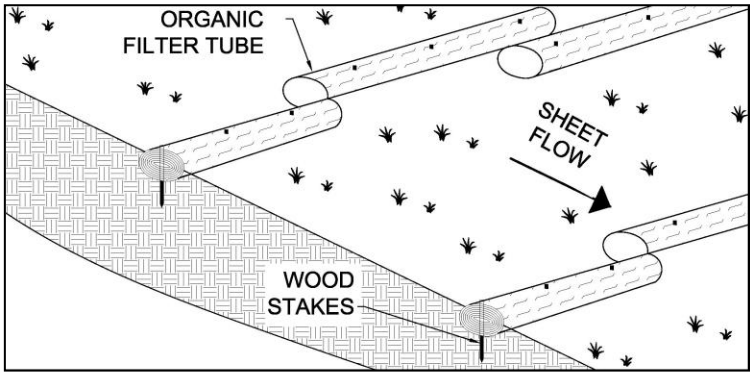
NOTE: DIMENSIONS OF THE BERM SHALL BE DESIGNED BASED ON FLOW CONDITIONS. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS TO DESIGN THE SWALE:

- SIZE OF CONTRIBUTING DRAINAGE AREA
- DESIGN STORM
- FLOW RATE
- BERM HEIGHT AND WIDTH

Figure 3.14 Schematics of Organic Filter Berm

3.6 Organic Filter Tubes

Sediment Control



Description: Organic filter tubes are comprised of an open weave, mesh tube that is filled with a filter material (compost, wood chips, straw, coir, aspen fiber, or a mixture of materials). The tube may be constructed of geosynthetic material, plastic, or natural materials. Organic filter tubes are also called fiber rolls, fiber logs, wattles, mulch socks, and/or coir rolls. Filter tubes detain flow and capture sediment as linear controls along the contours of a slope or as a perimeter control down-slope of a disturbed area.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Tube diameter when filled shall be specified on the plans
- 3 inch minimum embedment in soil
- 18 inch minimum overlap at ends of tubes
- Spacing based on drainage area and slope
- Must be staked on soil and secured with rockbags on pavement
- Turn ends of tube lines upslope a minimum of 10 feet

ADVANTAGES / BENEFITS:

- Effective means to treat sheet flow over a short distance
- Relatively easy to install
- May be used on steep slopes
- Can provide perimeter control on paved surfaces or where soil type prevents embedment of other controls
- Work well as perimeter controls around stockpiles

DISADVANTAGES / LIMITATIONS:

- Difficult to remove when wet and/or filled with sediment
- Relatively small effective areas for sediment capture

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Repair eroded areas underneath the organic filter tubes
- Re-align and stake tubes that are dislodged by flow
- Remove sediment before it reaches half the height of the exposed tube

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ◒ Oil & Grease
- ○ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.50-0.75
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ◒ Maintenance
- ○ Training
- ◒ Suitability for Slopes > 5%

Other Considerations:

- *None*

3.6.1 Primary Use

Organic filter tubes are long, flexible controls that are used along a line of constant elevation (along a contour) on slopes. They are used as perimeter controls down slope of disturbed areas, around temporary stockpiles and on side slopes where stormwater may runoff the area. The tubes maintain sheet flow, slow velocities, and capture sediment. When used in series on slopes, they also shorten the slope length and protect the slope from erosion.

3.6.2 Applications

Organic filter tubes include a wide variety of tube and filter materials. Organic filter tubes are used as a perimeter sediment barrier, similar to silt fence, for development projects and linear projects, such as roadways and utilities. They work well on individual residential lots and on lots being re-developed, where space may be limited. Organic filter tubes are most effective with coarse to silty soil types. Additional controls may be needed to remove fine silts and clay soils suspended in stormwater.

Organic filter tubes can be used on paved surfaces where it's not possible to stake a silt fence. Applications on paved surfaces include perimeter controls for soil stockpiles, pavement repair areas, utility trenching, and building demolition. When compost filter material is used in tubes on pavement, the material has the added benefit of removing some oil and grease from stormwater runoff.

Applications on slopes include temporary sediment control during construction and erosion control of the disturbed soil on the slope. Organic filter tubes may be used to control sheet flow on slopes when final stabilization measures are being applied and established.

Organic filter tubes may also be used for inlet protection and, in limited cases, as check dams in small drainage swales. Refer to *Section 3.4 Inlet Protection* and *Section 2.1 Check Dam* for the design criteria to use organic filter tubes in these applications.

3.6.3 Design Criteria

General Criteria

- Filter tubes should be installed along the contour.
- Tubes shall be staked with 2 inch by 2 inch wooden stakes at a maximum spacing of 4 feet. Rebar or similar metal stakes may be used instead of wooden stakes.
- When placed on pavement, sand or rock bags shall be placed abutting the down-slope side of the tubes to prevent runoff from dislodging the tubes. At a minimum, bags shall be placed one foot from each end of the tube and at the middle of the tube.
- Filter tubes shall be embedded a minimum of three inches when placed on soil. Placement on rock shall be designed as placement on pavement.
- The end of tubes shall overlap a minimum of 18 inches when multiple tubes are connected to form a linear control along a contour or a perimeter.
- Loose mulch material shall be placed against the log on the upstream side to facilitate contact with the ground.
- The last 10 feet (or more) at the ends of a line of tubes shall be turned upslope to prevent bypass by stormwater. Additional upslope lengths of tubes may be needed every 200 to 400 linear feet, depending on the traverse slope along the line of tubes.
- The most common sizes of tubes are 6 to 24 inches in diameter; however, tubes are available in sizes as small as 4 inches and up to 36 inches in diameter. The designer shall specify a diameter based on the site application. Tubes less than 8 inches in diameter when filled will require more frequent maintenance if used.

- Manufactured organic filter tube products shall have documentation of a minimum 75 percent soil retention using ASTM D7351 Standard Test Method for Determination of Sediment Retention Device Effectiveness in Sheet Flow Applications.
- When using manufactured tubes, the manufacturer's recommendations for diameter and spacing based on slope, flow velocities, and other site conditions shall be followed when they are more stringent than the design criteria in this section.
- When used as a perimeter control on grades of 10:1 or less, criteria in the following table shall be used as a guide for the size and installation rate of the organic filter tube.

Table 3.1 Perimeter Control Applications*

Drainage Area (Max)	*Max Flow Length to the Tube*	*Tube Diameter (Min)*
1/3 Acre per 100 feet	145 feet	18 inches
1/4 Acre per 100 feet	110 feet	15 inches
1/5 Acre per 100 feet	85 feet	12 inches
1/8 Acre per 100 feet	55 feet	9 inches

(Source: Modified and expanded from City of Plano Fact Sheet SP-13)
*Applicable on grades of 10:1 or flatter.

- When installing organic filter tubes along contours on slopes, criteria in the following table shall be used as a general guide for size and spacing of the tubes. Actual tube diameter and spacing shall be specified by the designer. The designer shall consider the tube manufacturers recommendations, the soil type, flow volume on the slope, required performance life, and erosion control measures that may be used in conjunction with the tubes.

Table 3.2 Maximum Spacing for Slope Protection

	Tube Diameter (Min)			
Slope (H:V)	*9 Inches*	*12 Inches*	*18 Inches*	*24 Inches*
5:1 to 10:1	35 feet	40 feet	55 feet	60 feet
4:1	30 feet	40 feet	50 feet	50 feet
3:1	25 feet	35 feet	40 feet	40 feet
2:1	20 feet	25 feet	30 feet	30 feet
1:1	10 feet	15 feet	20 feet	20 feet

(Source: Modified and expanded from Iowa Statewide Urban Design and Specifications Standards for Filter Socks)

Tube Material

- The designer shall specify the type of mesh based on the required life of the tube. At a minimum, the mesh shall have a rated life of one year under field conditions.
- If the tubes will be left onsite as part of the final stabilization, they must be constructed of 100 percent biodegradable jute, coir, sisal or similar natural fiber or 100 percent UV photodegradable plastic, polyester or geosynthetic material.
- Mesh tubes may be oval or round in cross-section.
- Mesh for the tubes shall be open and evenly woven. Size of weave openings shall be specified based on filter material. Openings may range from ½ inch for Erosion Control Compost to 2 inches for straw and coir.
- Mesh openings should not exceed ½ inch in diameter.

Filter Material

- Different filter materials have different properties and will affect sheet flow differently. The designer shall specify the type of material to be used (or excluded) on a particular site.
- Straw filter material shall be Certified Weed Free Forage. The straw must be in good condition, air-dried, and not rotten or moldy.
- Compost shall conform to the requirements for Erosion Control Compost in Item 161 of the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT 2004).
- Compost may provide some oil and grease removal; however, the large percentage of fines in compost will result in less filtering and more ponding of stormwater.
- Wood chips shall be 100 percent untreated chips and free of inorganic debris, such as plastic, glass, metal, etc. Wood chip size shall not be smaller than 1 inch and shall not exceed 3 inches in diameter. Shavings shall not be more than 5% of the total mass.

3.6.4 Design Guidance and Specifications

Specifications for Erosion Control Compost to be used as filter material may be found in Item 161 of the Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (TxDOT 2004).

3.6.5 Inspection and Maintenance Requirements

Organic filter tubes should be inspected regularly (at least as often as required by the TPDES Construction General Permit). The filter tube should be checked to ensure that it is in continuous contact with the soil at the bottom of the embedment trench. Closely check for rill erosion that may develop under the filter tubes. Eroded spots must be repaired and monitored to prevent reoccurrence. If erosion under the tube continues, additional controls are needed.

Staking shall be checked to ensure that the filter tubes are not moving due to stormwater runoff. Repair and re-stake slumping filter tubes. Tubes that are split, torn or unraveling shall be repaired or replaced.

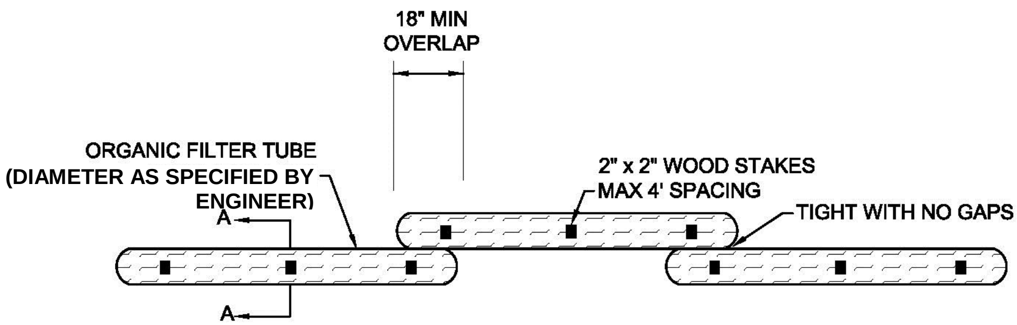
Check the filter tube material to make sure that it has not become clogged with sediment or debris. Clogged filter tubes usually lead to standing water behind the filter tube after the rain event. Sediment shall be removed from behind the filter tube before it reaches half the height of the exposed portion of the tube.

When sediment control is no longer needed on the site, the tubes may be split open and the filter material may be used for mulching during establishment of vegetation for final stabilization if it meets the criteria in *Section 2.5 Mulching*.

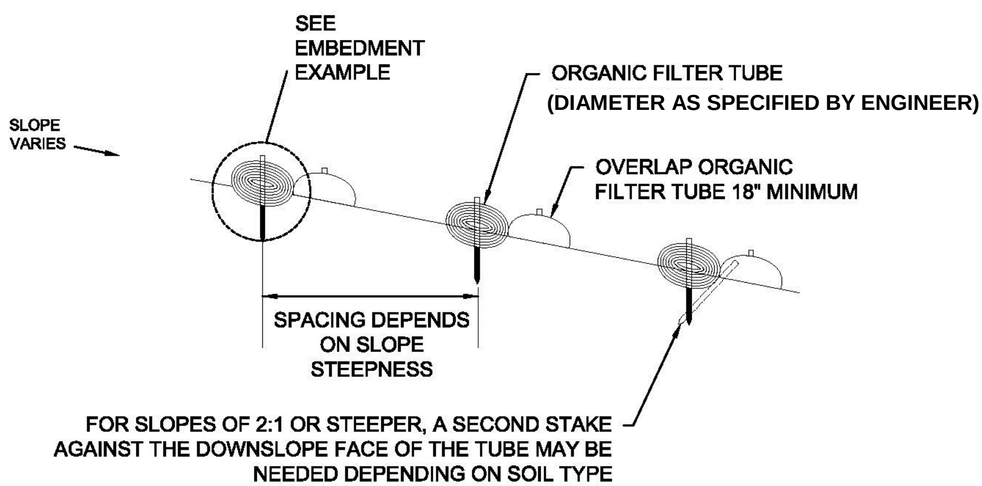
3.6.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



ORGANIC FILTER TUBE PERIMETER CONTROL PLAN VIEW
N.T.S.

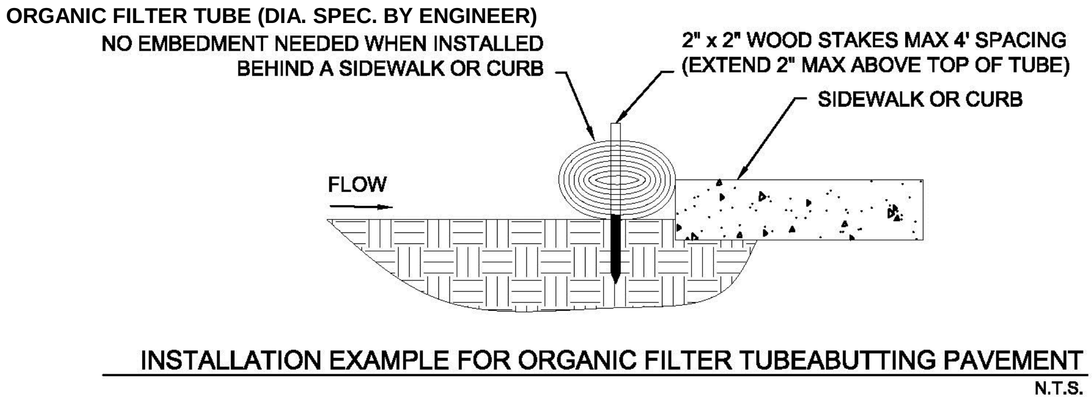


ORGANIC FILTER TUBE SLOPE PROTECTION PROFILE
N.T.S.

NOTES:

1. TYPE OF NETTING, FILTER MATERIAL, DIAMETER OF TUBE, AND SPACING OF TUBES SHALL BE SPECIFIED BY THE DESIGNER BASED ON THE FOLLOWING SITE PARAMETERS.
   - SIZE OF CONTRIBUTING DRAINAGE AREA
   - STEEPNESS OF SLOPE
   - GROUND CONDITIONS (SOIL OR PAVEMENT)
2. DESIGNER SHALL SHOW ON THE DRAWINGS THE LOCATIONS WHERE TUBES ARE TO BE TURNED UPSLOPE. UPSLOPE LENGTHS SHALL BE MINIMUM OF 10 FEET.

Figure 3.15 Schematics of Organic Filter Tubes



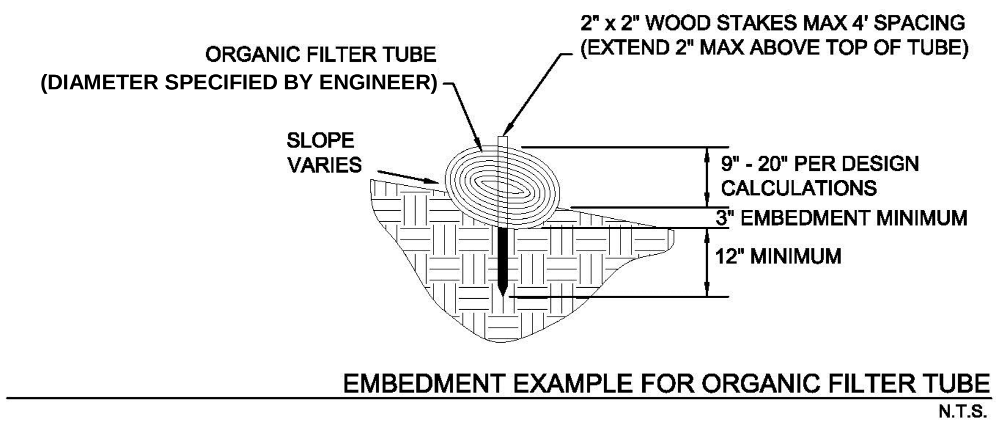
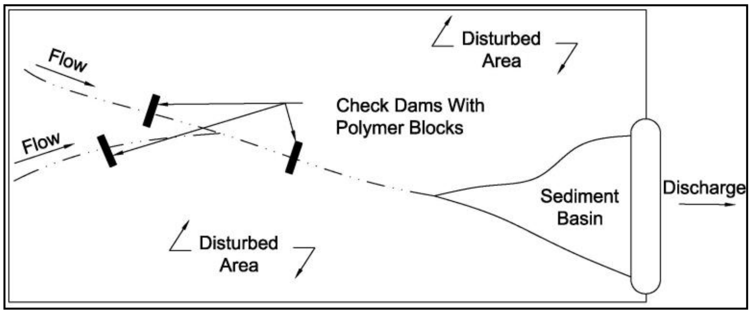


Figure 3.16 Examples of Organic Filter Tube Installation Methods

3.7 Passive Treatment System (PTS)

Sediment Control



Description: Passive Treatment Systems (PTS) consist of adding polymers to traditional sediment controls. The polymers act as a coagulant to cause flocculation of fine silts and clay soil particles that are not typically removed by the traditional controls. PTS devices include polymer gel socks, floc blocks, floc logs, and surface applications of powder or liquid polymers.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Install in flowing water upstream of sediment barriers
- Do not install at perimeter controls
- Select polymers based on soil type
- Closely monitor performance after storm events and adjust based on results

ADVANTAGES / BENEFITS:

- Less expensive and easier to operate than an ATS
- Capable of producing discharges with turbidity less than 280 NTU when applied and managed properly
- Improves removal of fine silt and clay particles from stormwater
- Reduces size requirements for a sediment basin
- May be used with dewatering devices

DISADVANTAGES / LIMITATIONS:

- Does not produce a predictable level of sediment removal
- Unknown levels of residual chemicals may be in discharges
- Trial and error often required to achieve high removal rates without off-site impacts

MAINTENANCE REQUIREMENTS:

- Inspect after every storm event
- Reapply and/or adjust locations after each storm event

TARGETED POLLUTANTS

● Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
$\boxed{\textbf{Sediment Barrier}}$
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.85

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
● Maintenance
◒ Training
◒ Suitability for Slopes > 5%

Other Considerations:

- *Potential off-site impacts of over dosing*

3.7.1 Primary Use

Passive treatment systems (PTS) are used to remove fine silt and clay soil particles, for which traditional sediment controls are the least effective. These fine particles are small enough to pass through the pores or void spaces of sediment barriers. They are also not removed by sediment basins, because their settling velocities require a detention time of days or weeks, not hours.

3.7.2 Applications

Passive treatment systems are applicable on construction sites that have a large percentage of fine silt and clay soils. The site must have an internal system of berms, swales and control devices where the PTS can be applied.

A PTS functions similarly to an active treatment system (ATS); however, it trades lower cost for less consistency in removal rates. PTSs are applicable on sites where variability in the effluent characteristics is acceptable, such as where the discharge from the PTS will flow through a vegetated area before leaving the site, instead of directly to receiving waters. A PTS may also be a viable alternative to an ATS when the concentration of suspended solids in runoff is relatively low due to soil type or good erosion control measures on the site.

The systems are also applicable where discharge criteria are established for a construction site or discharges from a disturbed area have the potential to impact special aquatic sites or sensitive receiving waters. Examples of sensitive receiving waters include wetlands regulated under Section 404 of the Clean Water Act, spring-fed water bodies, water bodies with species protected under the Federal Endangered Species Act or the State of Texas Threatened and Endangered Species Regulations, or water bodies closely monitored by citizen groups.

3.7.3 Design Criteria

The passive use of polymers to enhance sediment removal is a relatively new and rapidly evolving science. The pace of new product development is expected to accelerate due to the demand that is being driven by the Effluent Limitation Guidelines and Standards for the Construction and Development Point Source Category, issued by the EPA on December 1, 2009. The following criteria are general guidelines. It is essential that the designer of controls for a construction activity develop the PTS specifications based on consultation with technical experts at the company supplying the polymer.

General

- Polymers are used for PTS function by altering the charge of soil particles to allow them to floc, or “clump” together. The flocs are then trapped as a soil mass by a traditional sediment control, instead of passing through pores or voids of the control as a suspended particle. This effect will more quickly clog the sediment barrier and require more frequent cleaning.
- Polymers are available in anionic (negatively charged), non-ionic (no charge), and cationic (positively charged) forms. The charged state of the soil to be treated should be known to specify the proper polymer. Clay soils are typically anionic.
- Numerous types of polymers are commercially available; however, polyacrylamide (PAM) and chitosan are effective and non-toxic in a wide range of applications. They are the safest for use in systems that are not being continuously monitored.
- Polymers are available in numerous formulations that will have varying rates of effectiveness depending on the soil type being treated. Jar tests may be used to determine the effectiveness of a particular formulation or to evaluate different formulations if the one being used is not producing the desired results. Jar tests should be conducted in accordance with ASTM D2035 Standard Practice for Coagulation-Flocculation Jar Test of Water.

- PTSs may produce fluctuating and unpredictable levels of residual polymer in stormwater discharged from the site. Either residual testing or the use of an ATS is advisable when an endangered, threatened, or other sensitive species is present in the receiving water.
- Areas downstream of the PTS shall be monitored for floc accumulation. Design is partially trial and error. The goal is to provide sufficient polymer to produce onsite settling of soil flocs while not providing excess polymer that results in a chemical residual being discharged to receiving waters.

Floc Blocks, Floc Logs and Gel Socks

- Floc blocks and logs contain a solid form of polyacrylamide (PAM), a polymer that acts as a flocculating agent.
- Gel socks are a soft powder form of chitosan, a polymer that acts as a flocculating agent, contained within a fabric sock.
- The PTS should only be used in flowing water that is concentrated in swales or pipes. The turbulence of flowing water is necessary for mixing the polymer with the suspended soil.
- Swales and channels, upstream of a sediment basin, stone outlet sediment trap, check dam or other detention structure are effective locations for the PTS. This location gives the polymer time to mix before velocities are slowed by the sediment control, where the newly formed flocs can be settled or filtered.
- Removal rates increase proportionally with the distance the PTS is installed upstream of the sediment barrier. Longer distances correlate to higher removal rates.
- The PTS should be secured in a non-biodegradable mesh bag or galvanized wire cage, which in turn is securely anchored in a swale, channel, or pipe.
- The PTS should be installed in a manner that elevates above the ground at least six inches to minimize the potential for it to be in standing water a prolonged period of time.
- During long periods (weeks) of no precipitation, the floc blocks or logs that contain PAM may degrade from exposure to air and sunlight. In these situations, the blocks or logs should be replaced before the next predicted storm event. Alternatively, they may be removed during drought conditions to prevent their degradation, and then re-installed at the first forecast of precipitation.

Powder or Liquid Polymer

- Powder or liquid polymer can be sprayed onto check dams, silt fences, organic filter tubes, and other permeable barriers. Polymer can also be sprayed onto filter fabric or erosion control blankets lining a swale. The polymer will mix with stormwater as it filters through of flows over the control.
- Polymer shall not be applied to perimeter controls, as this will result in flocs forming after the stormwater has been discharged from the site. Liquid polymer shall only be applied to sediment controls that are located within the disturbed areas and have a perimeter control or other sediment trap down slope to catch the flocs.
- Polymer should be re-applied after each storm event. If a long period passes between storm events, the polymer will break-down and should be re-applied.
- Liquid polymer may be injected into concentrated stormwater (swales, channels, etc.) upstream of sediment basins to improve the removal efficiency of the basin. The polymer is typically injected using a small metering pump that is calibrated for a pre-established dose based on the design flow for the temporary control design storm (2-year, 24-hour).
- Liquid polymer may also be injected into the pump intake of dewatering systems to provide a higher sediment removal rate for fine silt and clay soil particles. Criteria for dewatering are in *Section 3.3 Dewatering*.

3.7.4 Design Guidance and Specifications

No specification for construction of this item is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.7.5 Inspection and Maintenance Requirements

Passive treatment systems should be inspected regularly (at least as often as required by the TPDES Construction General Permit).

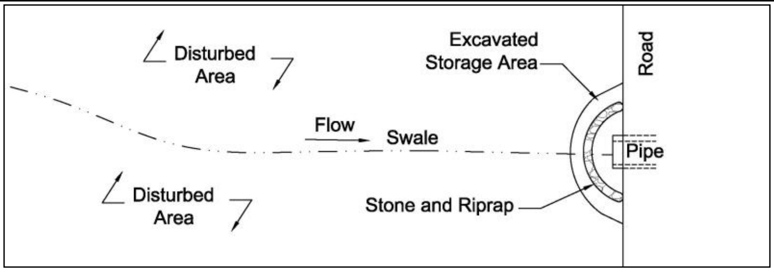
Floc blocks, floc logs, and gel socks should be checked after every storm event that produces stormwater runoff. Replace the PTS before it is completely dissolved. If the PTS is found to be submerged in standing water, it should be removed and re-installed at a new location where it will only be in contact with flowing stormwater.

The site's discharge points and downstream drainage infrastructure and water bodies should be inspected for accumulations of soil flocs. If flocs are found off the construction site, the PTS is not being implemented at a point where there is sufficient flow distance and time for polymer mixing and floc removal, or too much polymer is being used. The off-site floc accumulation must be removed if doing so will not negatively impact the receiving water. Then, the location or application of the PTS should be modified to provide additional mixing, more time for removal, or a lower dose, as applicable. If modifying the PTS is not possible, then an ATS may be needed to meet the discharge conditions for which a PTS was being used.

3.8 Pipe Inlet Sediment Trap

(Source: Modified from City of Plano BMP SP-11)

Sediment Control



Description: The pipe inlet sediment trap is a barrier surrounding a pipe inlet to capture sediment before it enters a closed drainage system. The barrier may be made of concrete block and filter stone or stone riprap and filter stone. The barrier provides both filtration and detention for sediment to settle in the excavated area.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Top of control shall be no higher than half the pipe diameter
- Excavate a storage volume for the 2-year, 24-hour design storm upslope of the barrier
- Side slopes of 2:1 or flatter on the excavated storage area
- Maximum drainage area of 5 acres
- Overflow capability required for large storm events

ADVANTAGES / BENEFITS:

- Removes sediment before it enters a closed conveyance system

DISADVANTAGES / LIMITATIONS:

- Ponding upstream of the pipe inlet with localized flooding possible
- Type A Pipe Inlet Sediment Trap limited to pipes of 36 inches in diameter or less

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove trash and debris after each storm event
- Remove sediment from the sediment storage area before it reaches half the design depth
- If de-watering of the storage volume is not occurring, clean or replace the filter stone

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ◒ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.50-0.75

(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ◒ Maintenance
- ○ Training
- ● Suitability for Slopes > 5%

Other Considerations:

- *Re-grading and stabilization of the control area after construction*

3.8.1 Primary Use

The pipe inlet sediment trap is used to intercept and filter sediment from concentrated flows at the inlet to a pipe. Capturing sediment before it enters a closed conveyance system decreases the cost of cleaning and removing sediment from the system.

3.8.2 Applications

The pipe inlet sediment trap should be used where existing or proposed storm drain pipes or culverts are used prior to final stabilization of the area draining to the pipe inlet.

3.8.3 Design Criteria

- The pipe inlet sediment trap must be designed with overflow capability, since this control is used where pipe culverts collect relatively heavy concentrations of stormwater flows.
- The drainage area contributing runoff to the sediment trap shall be not larger than 5 acres.
- Type A pipe inlet sediment trap is limited to pipes of 36 inches diameter and smaller. Type B pipe inlet sediment trap should be used on larger pipes.
- A stormwater and sediment storage area shall be excavated upslope of the stone barrier. Minimum storage area volume should be the volume of runoff from the temporary control design storm (2-year, 24-hour). Caution should be exercised during excavation so as to not undermine the control structure or the pipe that is being protected.
- Side slopes surrounding the storage area shall be 2:1 or flatter.
- Top of stone and sediment storage created by the stone shall not be any higher than half of the inlet pipe diameter. On Type A Pipe Inlet Sediment Trap, the concrete blocks shall not be stacked any higher than two blocks high.
- Concrete blocks shall be standard 8"x8"x16" concrete masonry units and shall be in accordance with ASTM C139, Concrete Masonry Units for Construction.
- Wire fabric shall be a standard galvanized hardware fabric with ½ inch by ½ inch openings.
- Filter stone shall be nominal 1½ inch washed stone with no fines. Angular shaped stone is preferable to rounded shapes.
- Stone riprap shall be 6 inch to 12 inch well-graded stone, Dry Riprap, Type A.
- Riprap shall be placed on filter fabric meeting the following minimum criteria:
    - Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles, 250-lbs.
    - Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 135-lbs.
    - Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 420-psi.
    - Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 20 (max).
- The pipe inlet sediment trap is most effective with coarse silt and sand soil particles. A passive treatment system may be used with the sediment trap to remove fine silt and clay soil particles.

3.8.4 Design Guidance and Specifications

Specifications for the riprap used in this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 803.3 Riprap.

3.8.5 Inspection and Maintenance Requirements

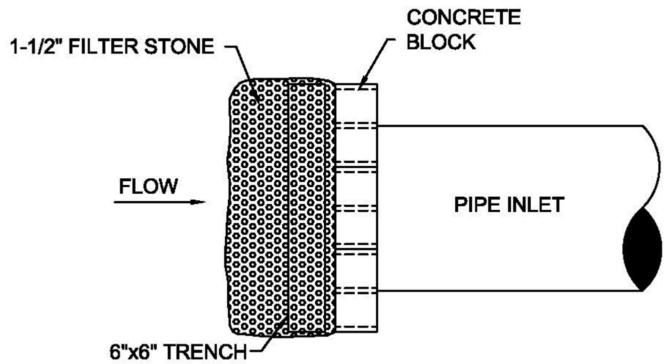
The pipe inlet sediment trap should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to ensure that the device is functioning properly. The controls should also be checked after storm events to verify it's operating properly and to inspect for damages. Make repairs as needed.

Trash and debris should be removed from the trap after each storm event to prevent it from plugging the rock. Remove sediment from the storage area before the depth of sediment is half of the design depth. If the sediment storage area is not being de-watered, the filter stone surrounding the pipe inlet must be cleaned or replaced. Cleaning the filter stone surface the first few times by raking may be adequate. Repeated sediment build-up and clogging of the stone will require filter stone removal and replacement.

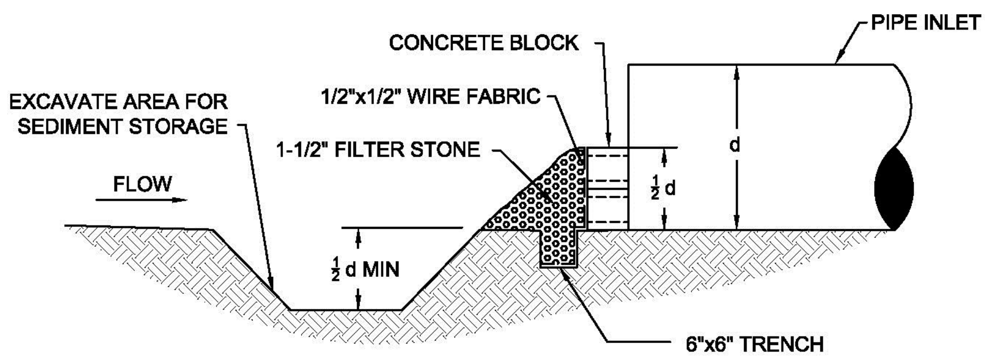
3.8.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



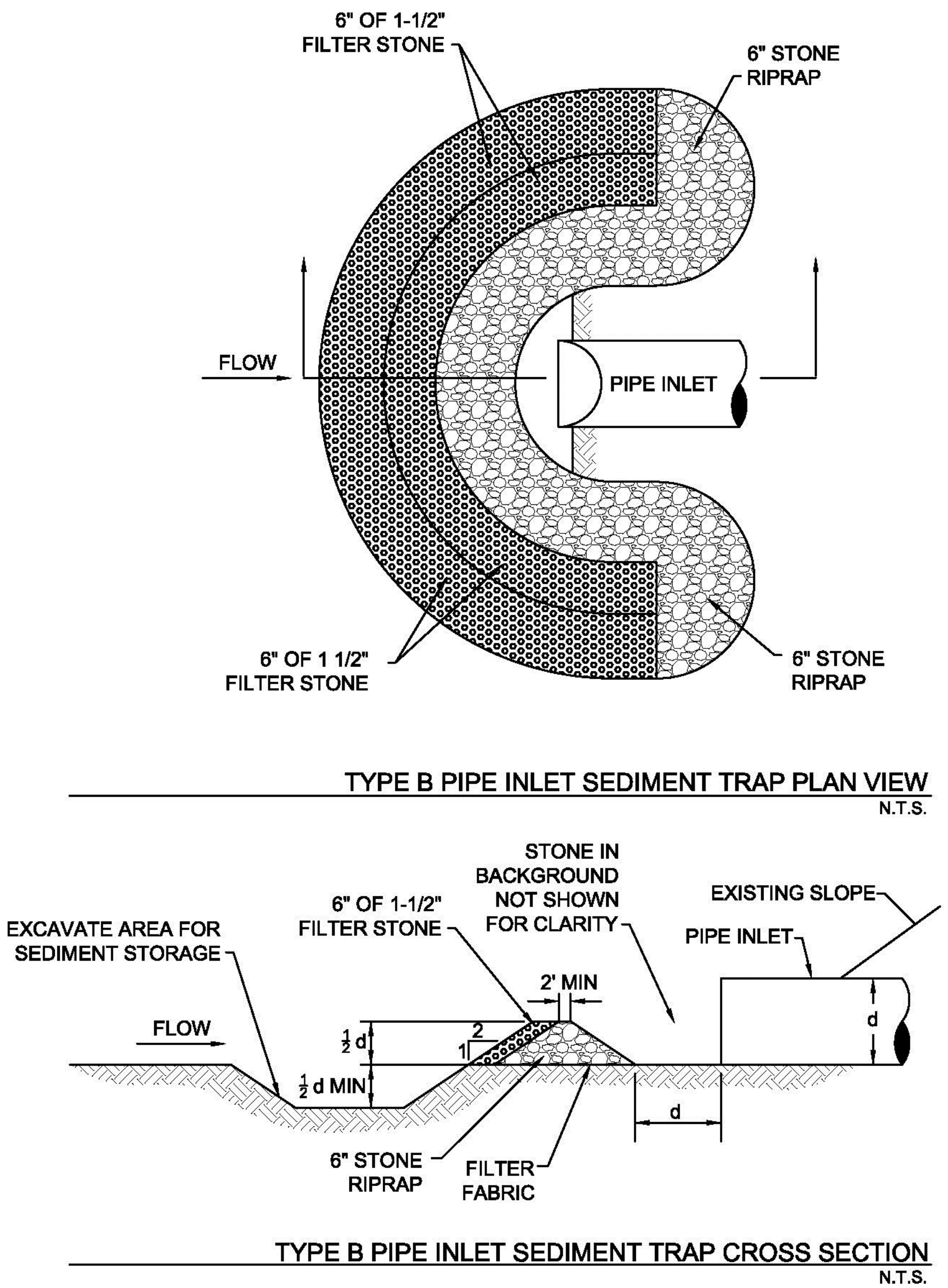
TYPE A PIPE INLET SEDIMENT TRAP PLAN VIEW
N.T.S.



TYPE A PIPE INLET SEDIMENT TRAP CROSS SECTION
N.T.S.

NOTE: THE TYPE A CONTROL IS FOR USE ON PIPES WITH A DIAMETER EQUAL TO OR LESS THAN 36 INCHES.

Figure 3.17 Schematics of Type A Pipe Inlet Sediment Trap
(Source: Modified from City of Plano BMP SP-11)

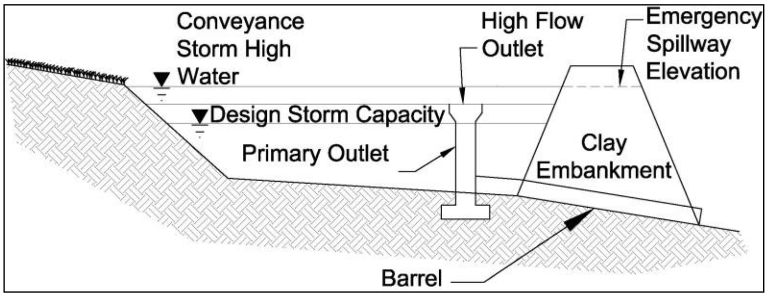


NOTE: THE TYPE B CONTROL IS FOR USE ON PIPES WITH A DIAMETER GREATER THAN 36 INCHES.

Figure 3.18 Schematics of Type B Pipe Inlet Sediment Trap
(Source: Modified from City of Plano BMP SP-11)

3.9 Sediment Basin

Sediment Control



Description: A sediment basin is an embankment with a controlled outlet that detains stormwater runoff, resulting in the settling of suspended sediment. The basin provides treatment for the runoff as well as detention and controlled release of runoff, decreasing erosion and flood impacts downstream.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum 4:1 length to width ratio
- Maximum embankment height and storage capacity limited by TCEQ requirements
- Minimum dewatering time of 36 hours
- Safely pass 25-year, 24-hour storm event without structure damage

ADVANTAGES / BENEFITS:

- Effective at removing suspended sand and loam
- May be both a temporary and permanent control
- Can be used in combination with passive treatment

DISADVANTAGES / LIMITATIONS:

- Effectiveness depends on type of outlet
- Limited effectiveness in removing fine silt and clay
- May require a relatively large portion of the site
- Storm events that exceed the design storm event may damage the structure and cause downstream impacts

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove obstructions from discharge structures
- Remove sediment and re-grade basin when storage capacity reduced by 20 percent

TARGETED POLLUTANTS

● Sediment
◒ Nutrients & Toxic Materials
○ Oil & Grease
◒ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
$\boxed{\text{Sediment Barrier}}$
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.50-0.90
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

● Capital Costs
◒ Maintenance
○ Training
● Suitability for Slopes > 5%

Other Considerations:

- *Public safety*
- *Mosquito breeding habitat*
- *Requires comprehensive planning and design*

3.9.1 Primary Use

Sediment basins should be used for all sites with adequate open space for a basin and where the site topography directs a majority of the site drainage to one point. Sediment basins are necessary as either temporary or permanent controls for sites with disturbed areas of 10 acres and larger that are part of a common drainage area unless specific site conditions limit their use.

3.9.2 Applications

Sediment basins serve as treatment devices that can be used on a variety of project types. They are normally used in site development projects in which large areas of land are available for the basin, a minor stream or off-line drainage way crosses the site, or a specific water feature is planned for the site. Sediment basins are highly effective at reducing sediment and other pollutants for design storm conditions. Sediment basins are typically easier to maintain than other structural controls (e.g. silt fences, etc).

A sediment basin by itself does not typically remove a sufficient percentage of fine silts and clays to be an effective sediment barrier. Table 3.3 provides a summary of sediment basin effectiveness based on soil type.

Table 3.3 Sediment Basin Effectiveness for Different Soil Types

Soil Type	*Runoff Potential*	*Settling Rate*	*Sediment Basin Effectiveness*	*Efficiency Rating (Fe)*
Sand	Low	High	High	0.90
Sandy Loam	Low	High	High	0.90
Sandy Silt Loam	Moderate	Moderate	Moderate	0.75
Silt Loam	Moderate	Moderate	Moderate	0.75
Silty Clay Loam	Moderate	Low	Low	0.75
Clay Loam	Great	Low	Low	0.50
Clay	Great	Low	Low	0.50

(Source: Michigan Department of Environmental Quality Soil Erosion and Sedimentation Control Training Manual)

When the disturbed area contains a high percentage of fine silt or clay soil types, the sediment basin may be used with a passive or active treatment system to remove these finer suspended solids. Design criteria may be found in *Section 3.1 Active Treatment System* and *Section 3.7 Passive Treatment System*.

3.9.3 Design Criteria

Texas Administrative Code Title 30, Chapter 299 (30 TAC 299), Dams and Reservoirs, contains specific requirements for dams that:

- Have a height greater than or equal to 25 feet and a maximum storage capacity greater than or equal to 15 acre-feet; or
- Have a height greater than six feet and a maximum storage capacity greater than or equal to 50 acre feet.

If the size of the detention basin meets or exceeds the above applicability, the design must be in accordance with state criteria, and the final construction plans and specifications must be submitted to the TCEQ for review and approval.

The following design criteria are for temporary sediment basins that are smaller than the TCEQ thresholds. The sediment basin shall be designed by a licensed engineer in the State of Texas. The criteria and schematics are the minimum and, in some cases, only concept level. It is the responsibility of the engineer to design and size the embankment, outfall structures, overflow spillway, and downstream energy dissipaters and stabilization measures. Alternative designs may be acceptable if submitted to the reviewing municipality with supporting design calculations.

Sediment Basin Location and Planning

- Design of the sediment basin should be coordinated with design of the permanent drainage infrastructure for the development.
- The basin shall not be located within a mapped 100-year floodplain unless its effects on the floodplain are modeled, and the model results are approved by the reviewing municipality.
- Basins shall not be located on a live stream that conveys stormwater from upslope property through the construction site.
- Basins may be located at the discharge point of a drainage swale that collects runoff from construction activities, or the basin may be located off-channel with a swale or dike constructed to divert runoff from disturbed areas to the basin. Design criteria for these controls are in *Section 2.2 Diversion Dike* and *Section 2.4 Interceptor Swale*.
- Sediment basins must be designed, constructed, and maintained to minimize mosquito breeding habitats by minimizing the creation of standing water.
- Temporary stabilization measures should be specified for all areas disturbed to create the basin.

Basin Size

- Minimum capacity of the basin shall be the calculated volume of runoff from a 2-year, 24-hour duration storm event plus sediment storage capacity of at least 1,000 cubic feet.
- The basin must be laid out such that the effective flow length to width ratio of the basin is a minimum of 4:1. Settling efficiencies are dependent on flow velocity, basin length, and soil type. Smaller particle sizes require slower velocities and longer basins. Basin dimensions should be designed based on flow velocities and anticipated particle sizes.
- Stoke’s equation for settling velocities, as modified to Newton’s equation for turbulent flow, may be used to estimate length required based on depth of the basin.

  Settling Velocity (ft/s) = 1.74 $[(\rho_p - \rho)gd/\rho]^{1/2}$ **(3.1)**

  Where:

  ρ_p = density of particles (lb/ ft^3)

  ρ = density of water (lb/ft^3)

  g = gravitational acceleration (ft/s^2)

  d = diameter of particles (ft)

- The effective length of sediment basins may be increased with baffles. Baffles shall be spaced at a minimum distance of 100 feet. Spacing should be proportional to the flow rate, with greater spacing for higher flow rates. Check the flow velocity in the cross section created by the baffles to ensure settling will occur.
- Baffles may be constructed by using excavated soil to create a series of berms within the basin; however, porous baffles are recommended. Porous baffles may consist of coir fiber, porous geotextiles, porous turbidity barriers, and similar materials. Porous materials disrupt the flow patterns, decrease velocities, and increase sedimentation.
- Basins have limited effectiveness on suspended clay soil particles. The basin’s length to width ratio typically should be 10:1 to effectively remove suspended clay particles. The use of passive treatment systems can significantly reduce this ratio and improve removal rates. Criteria are in *Section 3.7 Passive Treatment System*.

Embankment

- Top width shall be determined by the engineer based on the total height of the embankment as measured from the toe of the slope on the downstream side.
- Embankment side slopes shall be 3:1 or flatter.
- The embankment shall be constructed with clay soil, minimum Plasticity Index of 30 using ASTM D4318 Standard Test for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
- Clay soil for the embankment shall be placed in 8 inch lifts and compacted to 95 percent Standard Proctor Density at optimum moisture content using ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort.
- The embankment should be stabilized with rock riprap or temporary vegetation.

Outlet and Spillway

- The primary outlet shall have a minimum design dewatering time of 36 hours for the temporary control design storm (2-year, 24-hour).
- Whenever possible, the outlet shall be designed to drain the basin in less than 72 hours to minimize the potential for breeding mosquitoes.
- The basin's primary outlet and spillway shall be sized to pass the difference between the conveyance storm (25-year, 24-hour) and the temporary control design storm without causing damage to the embankment and structures.
- Unless infeasible, the primary outlet structure should withdraw water from the surface of the impounded water. Outlet structures that do this include surface skimmers, solid risers (non-perforated), flashboard risers, and weirs.
- Surface skimmers use a floating orifice to discharge water from the basin. Skimmers have the advantage of being able to completely drain the detention basin. Skimmers typically result in the greatest sediment removal efficiency for a basin, because they allow for a slower discharge rate than other types of surface outlets. Due to this slower discharge rate, a high flow riser may still be needed to discharge the conveyance storm if a large enough spillway is not feasible due to site constraints.
- Discharge rates for surface skimmers are dependent on the orifice configuration in the skimmer. Use manufacturer's flow rate charts to select the skimmer based on the flow rate needed to discharge the design storm from the basin within a selected time period (i.e. Q=Volume/time).
- Risers shall be designed using the procedures in *Section 3.9.7 Design Procedures*.
- Weir outlets should be designed using the guidance in *Section 2.2.2 of the Hydraulics Technical Manual*.
- Use of overflow risers and weirs result in a pool of water that should be accounted for in the design capacity of the basin. These outlet structures are good options when the temporary sediment basin will be retained as a permanent site feature upon completion of construction. If the basin is temporary and standing water is not acceptable during construction, the construction plans shall include procedures for dewatering the basin following criteria in *Section 3.3 Dewatering Controls*.
- Flashboard risers function like an overflow riser pipe, but they contain a series of boards that allow for adjustment of the pool level. The boards may be removed for draining the basin to a lower level. However, this operation can be difficult and a safety hazard when done manually.
- A perforated riser may be used as an outlet when surface discharge is not feasible. A perforated rise has the advantage of dewatering the basin; however, it also results in the lowest sediment removal efficiency. Perforated risers provide a relatively rapid drawdown of the pool, and they discharge water from the entire water column, resulting in more suspended sediment being discharged than with a surface outlet.

- Size and spacing of the orifices on a perforated riser shall be designed to provide the minimum detention time while allowing for the drawdown of detained water.
- Gravel (1½ to 3 inches) may be placed around the perforated riser to aid sediment removal, particularly the removal of fine soil particles, and to keep trash from plugging the perforations. The gravel is most effective when the basin will be used for less than a year. When installed for longer periods of time, the gravel may become clogged with fine sediments and require cleaning while submerged.
- The outlet of the outfall pipe (barrel) shall be stabilized with riprap or other materials designed using the conveyance storm flow rate and velocity. Velocity dissipation measures shall be used to reduce outfall velocities in excess of 5 feet per second.
- The outfall pipe through the embankment shall be provided with anti-seep collars connected to the exterior of the pipe section or at a normal joint of the pipe material. The anti-seep collar material shall be compatible with the pipe material used and shall have a watertight bond to the exterior of the pipe section. The size and number of collars shall be selected by the designer in accordance with the following formula and table:

  Collar Outside Dimension = X + Diameter of pipe in feet

  Example: Pipe Length = 45 feet
  Barrel Pipe Diameter = 12 inches = 1 foot
  2 anti-seep collars

  Anti-seep Collar Dimensions:
  3.4 feet (from table) + 1.0 foot (Pipe dia.) = 4.4 feet
  Use 2 anti-seep collars each being 4.4 feet square or 4.4 feet diameter if round.

Table 3.4 Number and Spacing of Anti-Seep Collars

	X Values - Feet			
Pipe Length	***Number of Anti-Seep Collars***			
	1	**2**	**3**	**4**
40	6.0	3.0		
45	6.8	3.4		
50	7.5	3.8	2.5	
55		4.2	2.8	
60		4.5	3.0	
65		4.9	3.3	
70		5.3	3.5	2.6
75		5.6	3.8	2.8
80		6.0	4.0	3.0

- Risers used to discharge high flows shall be equipped with an anti-vortex device and trash rack.
- Spillways shall be constructed in undisturbed soil material (not fill) and shall not be placed on the embankment that forms the basin.

3.9.4 Design Guidance and Specifications

Design guidance for temporary sediment basins is in *Section 3.9.7 Design Procedures*. Criteria for sediment basins that will become permanent detention basins are in *Section 3.6.3 of the iSWM Criteria Manual*. Additional design guidance for different types of outlet structures is in *Section 2.2 of the Hydraulics Technical Manual*.

No specification for construction of this item is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.9.5 Inspection and Maintenance Requirements

Sediment basins should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to check for damage and to insure that obstructions are not diminishing the effectiveness of the structure. Sediment shall be removed and the basin shall be re-graded to its original dimensions when the sediment storage capacity of the impoundment has been reduced by 20 percent. The removed sediment may be stockpiled or redistributed onsite in areas that are protected by erosion and sediment controls.

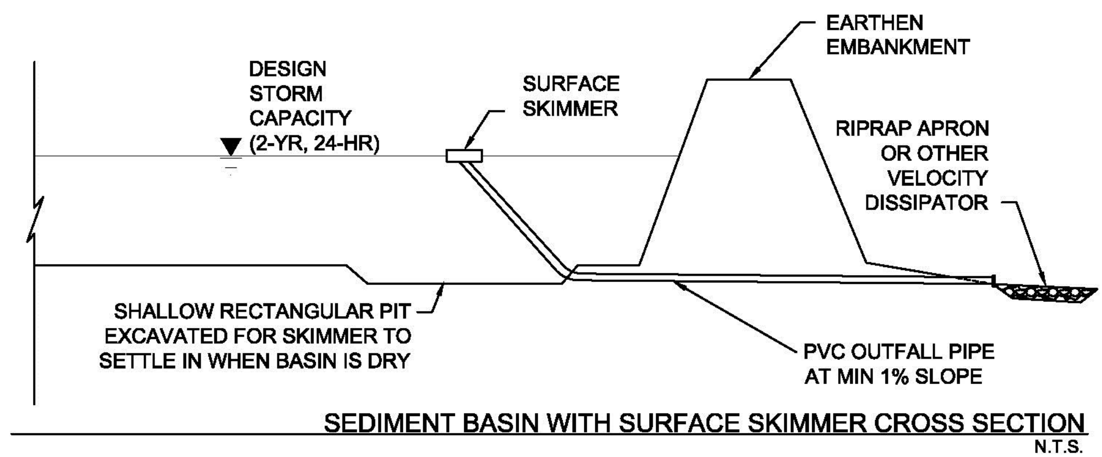
Inspect temporary stabilization of the embankment and graded basin and the velocity dissipaters at the outlet and spillway for signs of erosion. Repair any eroded areas that are found. Install additional erosion controls if erosion is frequently evident.

3.9.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. Dimensions of the sediment basin, embankment, and appurtenances shall be designed by an engineer licensed in the State of Texas. Construction drawings submitted to the municipality for review shall include, but are not limited to, the following information and supporting calculations.

- Embankment height, side slopes and top width.
- Dimensions of the skimmer, riser, weir or other primary outlet.
- Diameter of outfall pipe (barrel).
- Pool elevation for the temporary control design storm and conveyance storm.
- Outfall pipe flow rate and velocity for the temporary control design storm and conveyance storm.
- Spillway cross section, slope, flow rate, and velocity for the conveyance storm.
- Depth, width, length, and mean stone diameter for riprap apron or other velocity dissipation device at the outfall pipe and spillway discharge points.



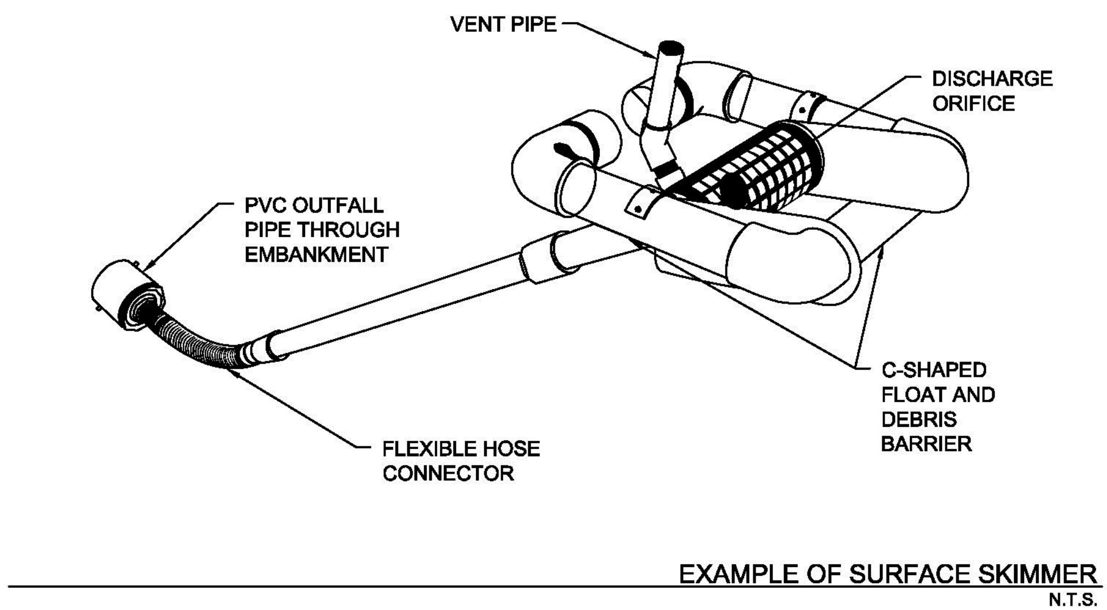


Figure 3.19 Schematics of Sediment Basin with Surface Skimmer

(Source: J.W. Faircloth & Son, Inc.)

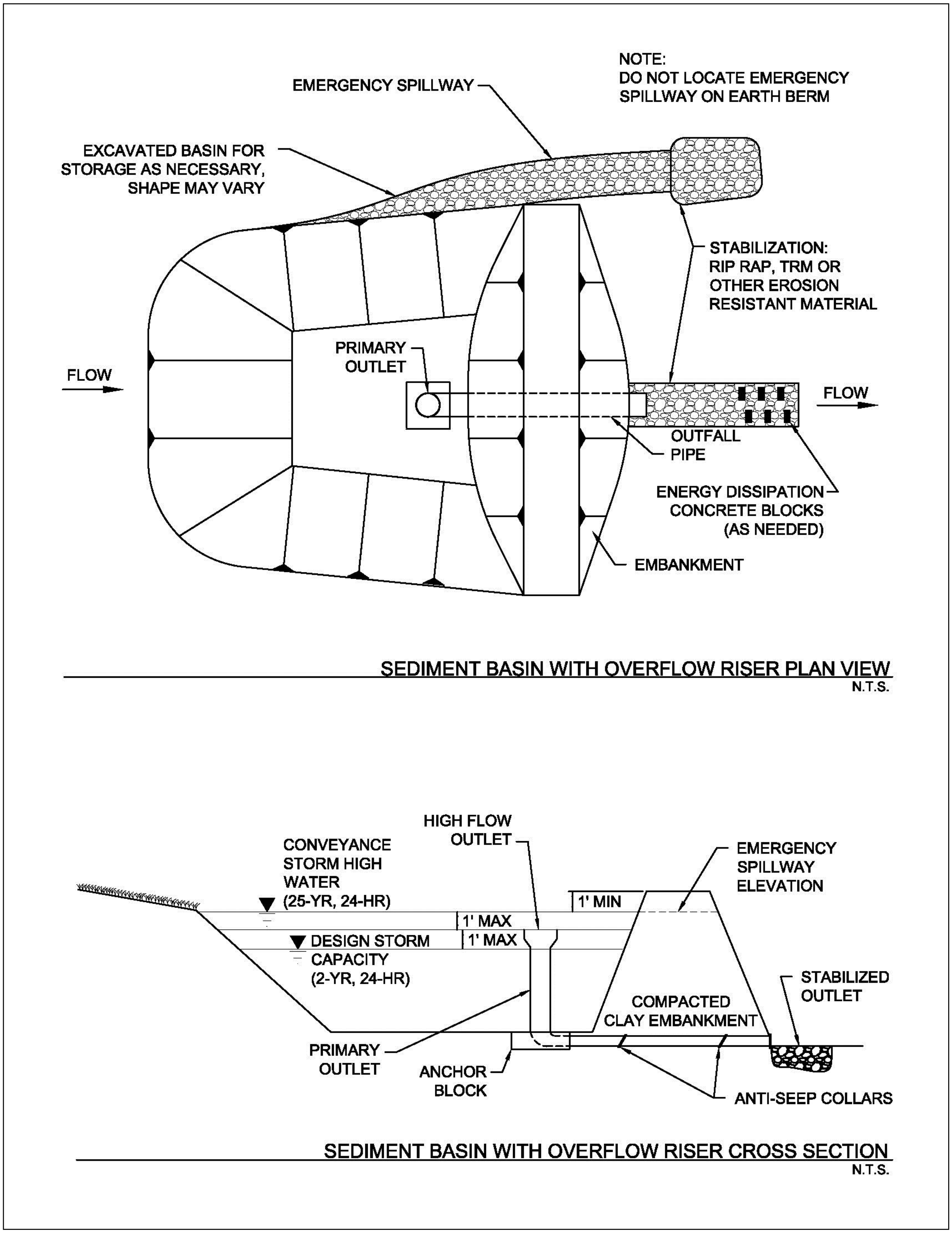
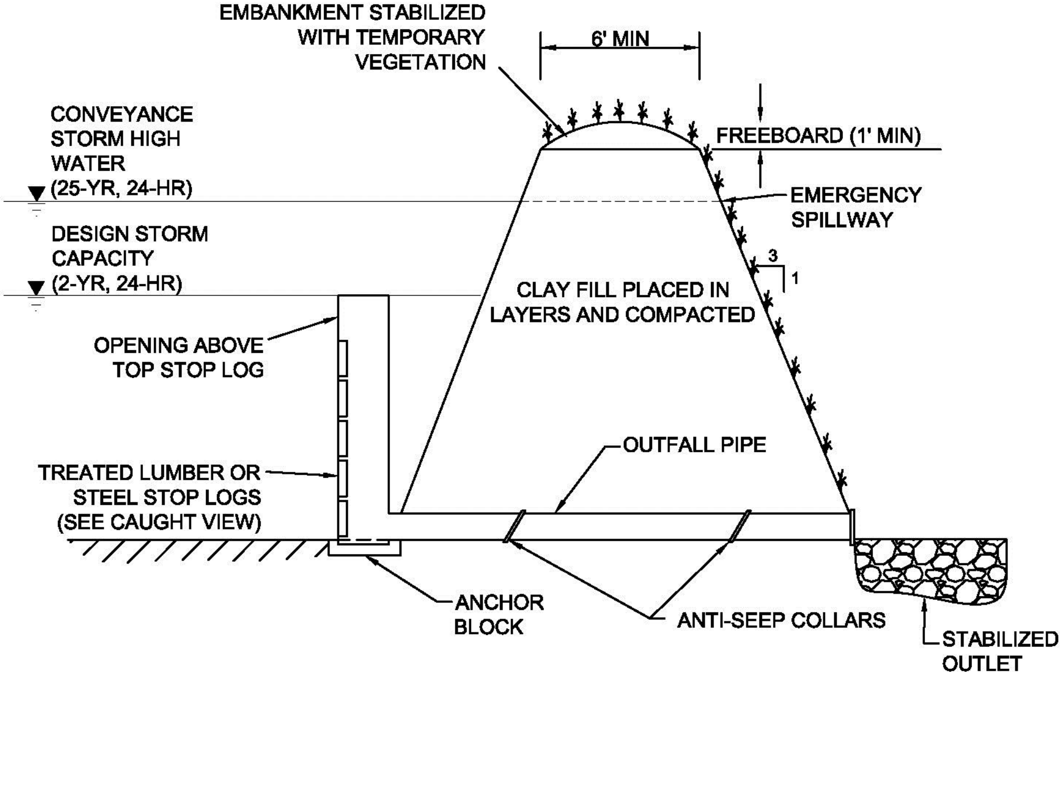
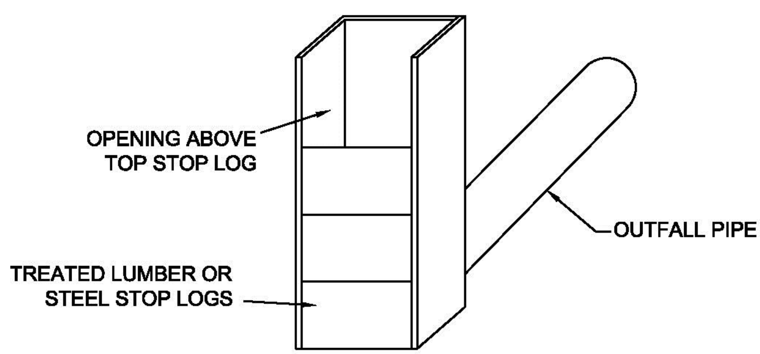


Figure 3.20 Schematics of Sediment Basin with Overflow Riser



EMBANKMENT CROSS SECTION WITH FLASHBOARD RISER
N.T.S.



FLASHBOARD RISER CONCEPT VIEW
N.T.S.

Figure 3.21 Schematics of Basin Embankment with Flashboard Riser

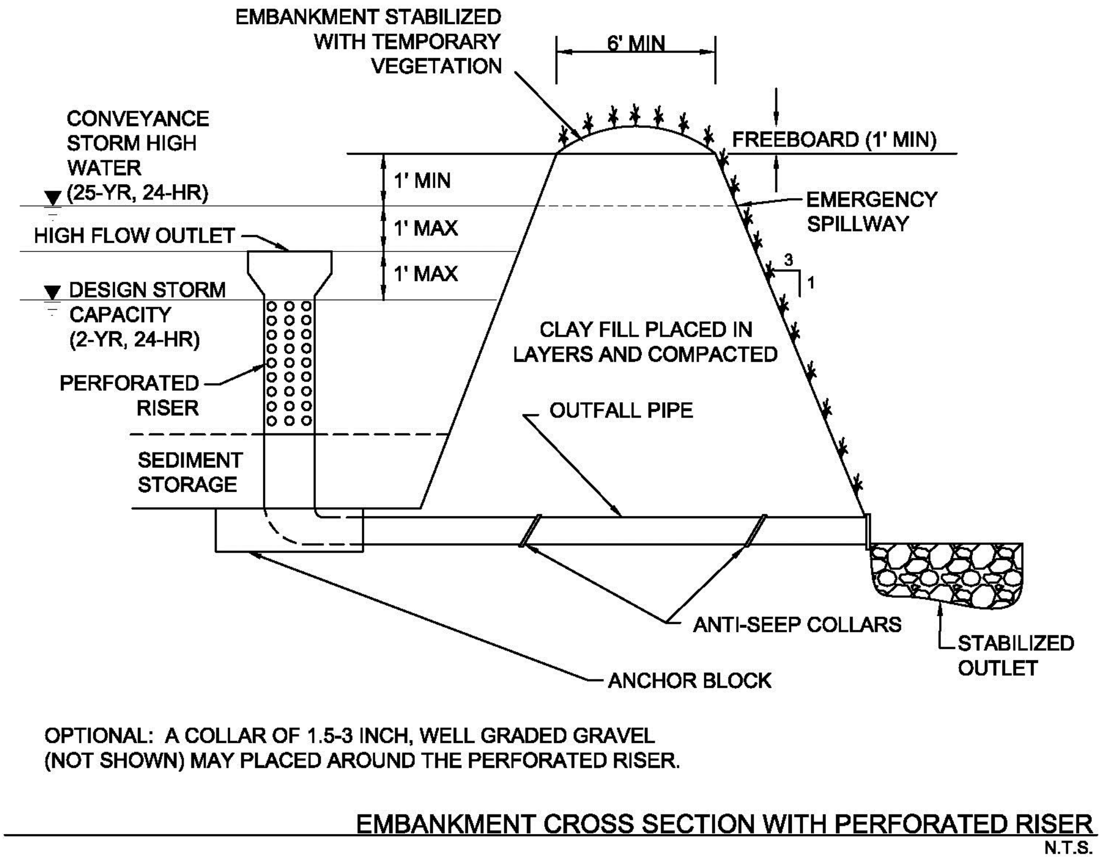


Figure 3.22 Schematic of Basin Embankment with Perforated Riser

3.9.7 Design Procedures

The following procedures provide a step-by-step method for the design of a temporary sediment basin that is smaller than the TCEQ thresholds for state requirements to apply. Criteria in *Section 3.8 of the iSWM Criteria Manual* should be used for the design of permanent basins (dry detention/extended dry detention) and stormwater ponds. *Section 3.9.8 Design Form* should be used to document the design values calculated for the temporary sediment basin.

These design procedures are provided as an example of the steps required to design a temporary sediment basin and are based on a specific type of primary outlet. When designing a sediment basin for a construction site, it's the engineer's responsibility to select the type of outlet that is appropriate based on criteria in the preceding sections and to modify the following procedures as needed to use appropriate calculations for the selected outlet, particularly in Steps 12, 13, and 14.

Step 1 Determine the required basin volume.

The basin volume shall be the calculated volume of runoff from the temporary control design storm (2-year, 24-hour) from each disturbed acre draining to the basin. When rainfall data is not available, a design volume of 3600 cubic feet of storage per acre drained may be used.

For a natural basin, the storage volume may be approximated as follows:

$$\mathbf{V_1 = 0.4 \times A_1 \times D_1} \qquad \mathbf{(3.2)}$$

where:

V_1 = the storage volume in cubic feet

A_1 = the surface area of the flooded area at the crest of the basin outlet, in square feet

D_1 = the maximum depth in feet, measured from the low point in the basin to the crest of the basin riser

Note 1: The volumes may be computed from more precise contour information or other suitable methods.

Note 2: Conversion between cubic feet and cubic yards is as follows:

Number of cubic feet x 0.037 = number of cubic yards

If the volume of the basin is inadequate or embankment height becomes excessive, pursue the use of excavation to obtain the required volume.

Step 2 Determine the basin shape.

The shape of the basin must be such that the length-to-width ratio is at least 4 to 1 according to the following equation:

$$\textbf{Length-to-width Ratio} = \frac{\mathbf{L}}{\mathbf{We}} \qquad \mathbf{(3.3)}$$

where:

We = A/L = the effective width

A = the surface area of the normal pool

L = the length of the flow path from the inflow to the outflow. If there is more than one inflow point, any inflow that carries more than 30 percent of the peak rate of inflow must meet these criteria.

The correct basin length can be obtained by proper site selection, excavation, or the use of baffles. Baffles increase the flow length by interrupting flow and directing it through the basin in a circuitous path to prevent short-circuiting. Porous baffles are recommended. Spacing of baffles should be wide enough to not cause a channeling effect within the basin. Analyze the flow cross section and velocity between baffles to ensure that velocities are not too fast for settling to occur.

Step 3 Design the embankment.

The side slopes of the embankment should be 3:1 or flatter.

Top width shall be determined by the engineer based on the total height of the embankment.

The area under the embankment should be cleared, grubbed, and stripped of topsoil to remove trees, vegetation, roots, or other objectionable materials. The pool area should also be cleared of all brush and trees.

The embankment fill material should be clay soil from an approved borrow area. It should be clean soil, free from roots, woody vegetation, oversized stones, and rocks.

Step 4 Select the type(s) of outlet(s).

The outlets for the basin may consist of a combination of a primary outlet and emergency spillway or a primary outlet alone. In either case, the outlet(s) must pass the peak runoff expected from the drainage area for the conveyance storm (25-year, 24-hour) without damage to the embankment, structures, or basin.

Step 5 Determine whether the basin will have a separate emergency spillway.

A side channel emergency spillway is required for sediment basins receiving stormwater from more than 10 acres.

Step 6 Determine the elevation of the crest of the basin outlet riser for the required volume.

Step 7 Estimate the elevation of the conveyance storm and the required height of the dam.

(a) If an emergency spillway is included, the crest of the basin outlet riser must be at least 1.0 foot below the crest of the emergency spillway.

(b) If an emergency spillway is included, the elevation of the peak flow through the emergency spillway (which will be the design high water for the conveyance storm) must be at least 1.0 foot below the top of embankment.

(c) If an emergency spillway is not included, the crest of the basin outlet riser must be at least 3 feet below the top of the embankment.

(d) If an emergency spillway is not included, the elevation of the design high water for the conveyance storm must be 2.0 feet below the top of the embankment.

Step 8 Determine the peak rate of runoff for a 25-year storm.

Using SCS TR 55 Urban Hydrology for Small Watersheds or other methods, determine the peak rate of runoff expected from the drainage area of the basin for the conveyance storm. The "C" factor or "CN" value used in the runoff calculation should be derived from analysis of the contributing drainage area at the peak of land disturbance (condition which will create greatest peak runoff).

Step 9 Design the basin outlet.

(a) If an emergency spillway is included, the basin outfall must at least pass the peak rate of runoff from the basin drainage area for the temporary control design storm (2-year, 24-hour).

Q_p = the 2-year peak rate of runoff.

(b) If an emergency spillway is not included, the basin outfall must pass the peak rate of runoff from the basin drainage area for the conveyance storm (25-year, 24-hour).

Q_{25} = the 25-year peak rate of runoff.

(c) Refer to Figure 3.23, where h is the difference between the elevation of the crest of the basin outlet riser and the elevation of the crest of the emergency spillway.

(d) Enter Figure 3.24 with Q_p. Choose the smallest riser which will pass the required flow with the available head, h.

(e) Refer to Figure 3.23, where H is the difference in elevation of the centerline of the outlet of the outfall and the crest of the emergency spillway. L is the length of the barrel through the embankment.

(f) Enter Table 3.5 or Table 3.6 with H. Choose the smallest size outlet that will pass the flow provided by the riser. If L is other than 70 feet, make the necessary correction.

(g) The basin riser shall consist of a solid (non-perforated), vertical pipe or box of corrugated metal joined by a watertight connection to a horizontal pipe (outfall) extending through the embankment and discharging beyond the downstream toe of the fill. Another approach is to utilize a perforated vertical riser section surrounded by filter stone.

(h) The basin outfall, which extends through the embankment, shall be designed to carry the flow provided by the riser with the water level at the crest of the emergency spillway. The connection between the riser and the outfall must be watertight. The outlet of the outfall must be protected to prevent erosion or scour of downstream areas.

(i) Weirs, skimmers and other types of outlets may be used if accompanied with appropriate calculations.

Step 10 Design the emergency spillway.

(a) The emergency spillway must pass the remainder of the 25-year peak rate of runoff not carried by the basin outlet.

(b) Compute: $Q_e = Q_{25} - Q_p$

(c) Refer to Figure 3.25 and Table 3.7.

(d) Determine approximate permissible values for b, the bottom width; s, the slope of the exit channel; and X, minimum length of the exit channel.

(e) Enter Table 3.7 and choose the exit channel cross-section which passes the required flow and meets the other constraints of the site.

(f) Notes:

1. The maximum permissible velocity for vegetated waterways must be considered when designing an exit channel.
2. For a given Hp, a decrease in the exit slope from S as given in the table decreases spillway discharge, but increasing the exit slope from S does not increase discharge. If an exit slope (Se) steeper than S is used, then the exit should be considered an open channel and analyzed using the Manning's Equation.
3. Data to the right of heavy vertical lines should be used with caution, as the resulting sections will be either poorly proportioned or have excessive velocities.

(g) The emergency spillway should not be constructed over fill material.

(h) The emergency spillway should be stabilized with rock riprap or temporary vegetation upon completion of the basin.

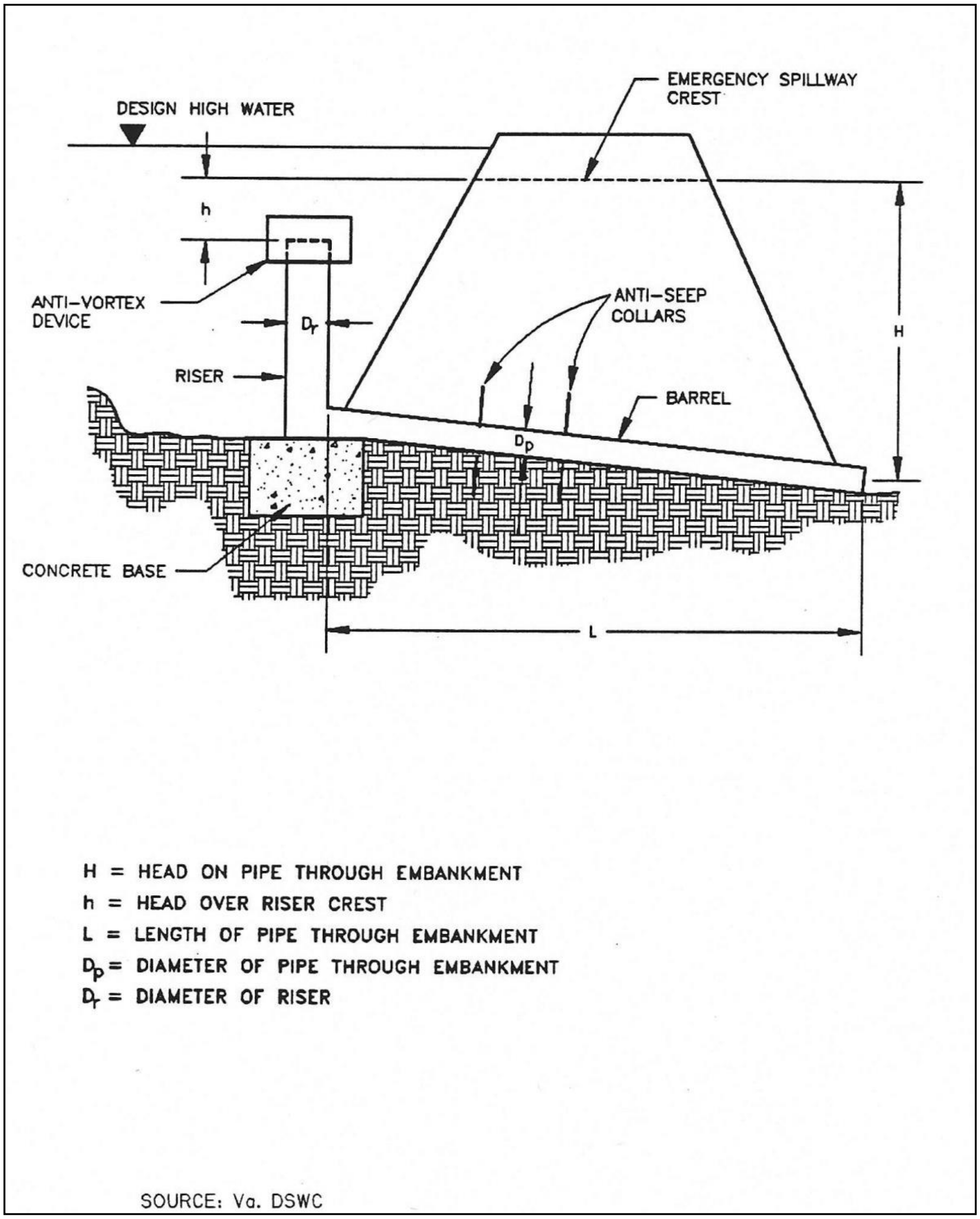


Figure 3.23 Example of Basin Outlet Design

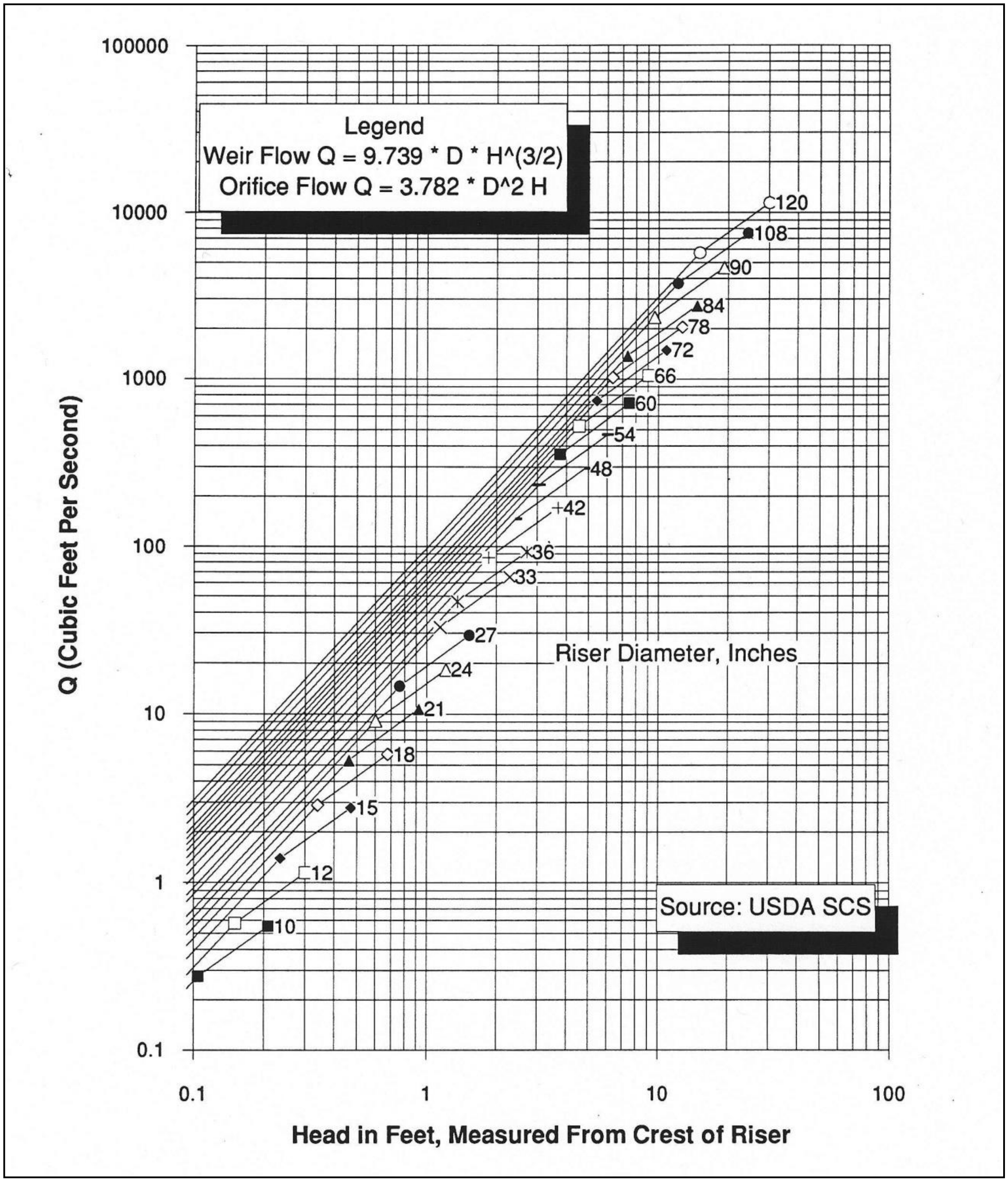


Figure 3.24 Riser Inflow Curves for Basin Outlet Design

Table 3.5 Pipe Flow Chart, n=0.013

For Reinforced Concrete Pipe Inlet Km = Ke + Kb =0.65 and 70 Feet of Reinforced Concrete Pipe Conduit (Full Flow Assumed)
Note: Correction Factors for pipe lengths other than 70 feet

Head	Pipe Diameter in Inches																	
(in feet)	12	15	18	21	24	30	36	42	48	54	60	66	72	78	84	90	96	102
1	3.22	5.44	8.29	11.8	15.9	26	38.6	53.8	71.4	91.5	114	139	167	197	229	264	302	342
2	4.55	7.69	11.7	16.7	22.5	36.8	54.6	76	101	129	161	197	236	278	324	374	427	483
3	5.57	9.42	14.4	20.4	27.5	45	66.9	93.1	124	159	198	241	289	341	397	458	523	592
4	6.43	10.9	16.6	23.5	31.8	52	77.3	108	143	183	228	278	334	394	459	529	604	683
5	7.19	12.2	18.5	26.3	35.5	58.1	86.4	120	160	205	255	311	373	440	513	591	675	764
6	7.88	13.3	20.3	28.8	38.9	63.7	94.6	132	175	224	280	341	409	482	562	647	739	837
7	8.51	14.4	21.9	31.1	42	68.8	102	142	189	242	302	368	441	521	607	699	798	904
8	9.1	15.4	23.5	33.3	44.9	73.5	109	152	202	259	323	394	472	557	685	748	854	966
9	9.65	16.3	24.9	35.3	47.7	78	116	161	214	275	342	418	500	590	688	793	905	1025
10	10.2	17.2	26.2	37.2	50.2	82.2	122	170	226	289	361	440	527	622	725	836	954	1080
11	10.7	18	27.5	39	52.7	86.2	128	178	237	304	379	462	553	653	761	877	1001	1133
12	11.1	18.9	28.7	40.8	55	90.1	134	186	247	317	395	482	578	682	794	916	1045	1184
13	11.6	19.6	29.9	42.4	57.3	93.7	139	194	257	330	411	502	601	710	827	953	1088	1232
14	12	20.4	31	44.1	59.4	97.3	145	201	267	342	427	521	624	736	858	989	1129	1278
15	12.5	21.1	32.1	45.6	61.5	101	150	208	277	354	442	539	646	762	888	1024	1169	1323
16	12.9	21.8	33.2	47.1	63.5	104	155	215	286	366	457	557	667	787	917	1057	1207	1367
17	13.3	22.4	34.2	48.5	65.5	107	159	222	294	377	471	574	688	812	946	1090	1244	1409
18	13.7	23.1	35.2	49.9	67.4	110	164	228	303	388	484	591	708	835	973	1121	1280	1450
19	14	23.7	36.1	51.3	69.2	113	168	234	311	399	497	607	727	858	1000	1152	1315	1489
20	14.4	24.3	37.1	52.6	71	116	173	240	319	409	510	623	746	880	1026	1182	1350	1528
21	14.7	24.9	38	53.9	72.8	119	177	246	327	419	523	638	764	902	1051	1211	1383	1566
22	15.1	25.5	38.9	55.2	74.5	122	181	252	335	429	535	653	782	923	1076	1240	1415	1603
23	15.4	26.1	39.8	56.5	76.2	125	186	258	342	439	547	668	800	944	1100	1268	1447	1639
24	15.8	26.7	40.6	57.7	77.8	127	189	263	350	448	559	682	817	964	1123	1295	1478	1674
25	16.1	27.2	41.5	58.9	79.4	130	193	269	357	458	571	696	834	984	1147	1322	1509	1708
26	16.4	27.7	42.3	60	81	133	197	274	364	467	582	710	850	1004	1169	1348	1539	1742
27	16.7	28.3	43.1	61.2	82.5	135	201	279	371	476	593	723	867	1023	1192	1373	1568	1775
28	17	28.8	43.9	62.3	84.1	138	204	285	378	484	604	737	883	1041	1214	1399	1597	1808
29	17.3	29.3	44.7	63.4	85.5	140	208	290	384	493	615	750	898	1060	1235	1423	1625	1840
30	17.6	29.8	45.4	64.5	87	142	212	294	391	501	625	763	913	1078	1256	1448	1653	1871
	Correction Factors for Other Pipe Lengths																	
20	1.3	1.24	1.21	1.18	1.15	1.12	1.1	1.08	1.07	1.06	1.05	1.05	1.04	1.04	1.03	1.03	1.03	1.03
30	1.22	1.18	1.15	1.13	1.12	1.09	1.08	1.06	1.05	1.05	1.04	1.04	1.03	1.03	1.03	1.02	1.02	1.02
40	1.15	1.13	1.11	1.1	1.08	1.07	1.05	1.05	1.04	1.03	1.03	1.03	1.02	1.02	1.02	1.02	1.02	1.02
50	1.09	1.08	1.07	1.06	1.05	1.04	1.04	1.03	1.03	1.02	1.02	1.02	1.02	1.01	1.01	1.01	1.01	1.01
60	1.04	1.04	1.03	1.03	1.03	1.02	1.02	1.02	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
70	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
80	0.96	0.97	0.97	0.97	0.98	0.98	0.98	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
90	0.93	0.94	0.94	0.95	0.95	0.96	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.99	0.99	0.99	0.99	0.99
100	0.9	0.91	0.92	0.93	0.93	0.95	0.95	0.96	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.98	0.99
120	0.84	0.86	0.87	0.89	0.9	0.91	0.93	0.94	0.94	0.95	0.96	0.96	0.96	0.97	0.97	0.97	0.97	0.98
140	0.8	0.82	0.83	0.85	0.96	0.88	0.9	0.91	0.92	0.93	0.94	0.94	0.95	0.95	0.96	0.96	0.96	0.97
160	0.76	0.78	0.8	0.82	0.83	0.86	0.88	0.89	0.9	0.91	0.92	0.93	0.94	0.94	0.95	0.95	0.95	0.96

Source: USDA SCS

Table 3.6 Pipe Flow Chart, n=0.025

For Corrugated Metal Pipe Inlet Km = Ke + Kb =0.65 and 70 Feet of Corrugated Metal Pipe Conduit (Full Flow Assumed)
Note: Correction Factors for pipe lengths other than 70 feet

Head (In feet)	Pipe Diameter in Inches 6	8	10	12	15	18	21	24	30	36	42	48	54	60	66	72	78	84	90	96	102
1	0.33	0.7	1.25	1.98	3.48	5.47	7.99	11	18.8	28.8	41.1	55.7	72.6	91.8	113	137	163	191	222	255	290
2	0.47	0.99	1.76	2.8	4.92	7.74	11.3	15.6	26.6	40.8	58.2	78.8	103	130	160	194	231	271	314	360	410
3	0.58	1.22	2.16	3.43	6.02	9.48	13.8	19.1	32.6	49.9	71.2	96.5	126	159	196	237	282	331	384	441	502
4	0.67	1.4	2.49	3.97	6.96	10.9	16	22.1	37.6	57.7	82.3	111	145	184	226	274	326	383	444	510	580
5	0.74	1.57	2.79	4.43	7.78	12.2	17.9	24.7	42.1	64.5	92	125	162	205	253	306	365	428	496	570	648
6	0.82	1.72	3.05	4.86	8.52	13.4	19.6	27	46.1	70.6	101	136	178	225	277	336	399	469	544	624	710
7	0.88	1.86	3.3	5.25	9.2	14.5	21.1	29.2	49.8	76.3	109	147	192	243	300	362	431	506	587	674	767
8	0.94	1.99	3.53	5.61	9.84	15.5	22.6	31.2	53.2	81.5	116	158	205	260	320	388	461	541	628	721	820
9	1	2.11	3.74	5.95	10.4	16.4	24	33.1	56.4	86.5	123	167	218	275	340	411	489	574	666	764	870
10	1.05	2.22	3.94	6.27	11	17.3	25.3	34.9	59.5	91.2	130	176	230	290	358	433	516	605	702	806	917
11	1.1	2.33	4.13	6.58	11.5	18.2	26.5	36.6	62.4	95.6	136	185	241	304	376	454	541	635	736	845	962
12	1.15	2.43	4.32	6.87	12.1	19	27.7	38.2	65.2	99.9	142	193	252	318	392	475	565	663	769	883	1004
13	1.2	2.53	4.49	7.15	12.6	19.7	28.8	39.8	67.8	104	148	201	262	331	408	494	588	690	800	919	1045
14	1.25	2.63	4.66	7.42	13	20.5	29.9	41.3	70.4	108	154	208	272	343	424	513	610	716	830	953	1085
15	1.29	2.72	4.83	7.68	13.5	21.2	30.9	42.8	72.8	112	159	216	281	355	439	531	631	741	860	987	1123
16	1.33	2.81	4.99	7.93	13.9	21.9	32	44.2	75.2	115	165	223	290	367	453	548	652	765	888	1019	1160
17	1.37	2.9	5.14	8.18	14.3	22.6	32.9	45.5	77.5	119	170	230	300	378	467	565	672	789	915	1051	1195
18	1.41	2.98	5.29	8.41	14.8	23.2	33.9	46.8	79.8	120	174	236	308	389	480	581	692	812	942	1081	1230
19	1.45	3.06	5.43	8.64	15.2	23.9	34.8	48.1	82	126	179	243	316	400	494	597	711	834	967	1111	1264
20	1.49	3.14	5.57	8.87	15.6	24.5	35.7	49.4	84.1	129	184	249	325	410	506	613	729	856	993	1139	1297
21	1.53	3.22	5.71	9.09	15.9	25.1	36.6	50.6	86.2	132	188	255	333	421	519	628	747	877	1017	1168	1329
22	1.56	3.29	5.85	9.3	16.3	25.7	37.5	51.8	88.2	135	193	261	341	430	531	643	765	898	1041	1195	1360
23	1.6	3.37	5.98	9.51	16.7	26.2	38.3	53	90.2	138	197	267	348	440	543	657	782	918	1064	1222	1390
24	1.63	3.44	6.11	9.72	17	26.8	39.1	54.1	92.1	141	201	273	356	450	555	671	799	937	1087	1248	1420
25	1.66	3.51	6.23	9.92	17.4	27.4	39.9	55.2	94	144	206	279	363	459	566	685	815	957	1110	1274	1450
26	1.7	3.58	6.36	10.1	17.7	27.9	40.7	56.3	95.9	147	210	284	370	468	577	699	831	976	1132	1299	1478
27	1.73	3.65	6.48	10.3	18.1	28.4	41.5	57.4	97.7	150	214	290	377	477	588	712	847	994	1153	1324	1507
28	1.76	3.72	6.6	10.5	18.4	29	42.3	58.4	99.5	153	218	295	384	486	599	725	863	1013	1174	1348	1534
29	1.79	3.78	6.71	10.7	18.7	29.5	43	59.5	101	155	221	300	391	494	610	738	878	1030	1195	1372	1561
30	1.82	3.85	6.83	10.9	19.1	30	43.7	60.5	103	158	225	305	398	503	620	750	893	1048	1216	1396	1588
	Correction Factors for Other Pipe Lengths																				
20	1.69	1.63	1.58	1.53	1.47	1.42	1.37	1.34	1.28	1.24	1.2	1.18	1.16	1.14	1.13	1.11	1.1	1.1	1.09	1.08	1.08
30	1.44	1.41	1.39	1.36	1.32	1.29	1.27	1.24	1.21	1.18	1.15	1.13	1.12	1.11	1.1	1.09	1.08	1.07	1.07	1.06	1.06
40	1.28	1.27	1.25	1.23	1.21	1.2	1.18	1.17	1.14	1.12	1.11	1.1	1.09	1.08	1.07	1.06	1.06	1.05	1.05	1.05	1.04
50	1.16	1.16	1.15	1.14	1.13	1.12	1.11	1.1	1.09	1.08	1.07	1.06	1.06	1.05	1.05	1.04	1.04	1.04	1.03	1.03	1.03
60	1.07	1.07	1.07	1.06	1.06	1.05	1.05	1.05	1.04	1.04	1.03	1.03	1.03	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.01
70	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
80	0.94	0.94	0.95	0.95	0.95	0.95	0.96	0.96	0.96	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.98	0.99	0.99	0.99
90	0.89	0.89	0.9	0.9	0.91	0.91	0.92	0.92	0.93	0.94	0.94	0.95	0.95	0.96	0.96	0.96	0.97	0.97	0.97	0.97	0.94
100	0.85	0.85	0.86	0.86	0.87	0.88	0.89	0.89	0.9	0.91	0.92	0.93	0.93	0.94	0.94	0.95	0.95	0.95	0.96	0.96	0.94
120	0.78	0.79	0.79	0.8	0.81	0.82	0.83	0.83	0.85	0.86	0.87	0.89	0.89	0.9	0.91	0.91	0.92	0.93	0.93	0.94	0.94
140	0.72	0.73	0.74	0.75	0.76	0.77	0.78	0.79	0.81	0.82	0.84	0.85	0.86	0.87	0.88	0.86	0.89	0.9	0.91	0.91	0.92
160	0.68	0.69	0.69	0.7	0.71	0.73	0.74	0.75	0.77	0.79	0.8	0.82	0.83	0.84	0.85	0.92	0.87	0.88	0.89	0.89	0.9

Source: USDA SCS

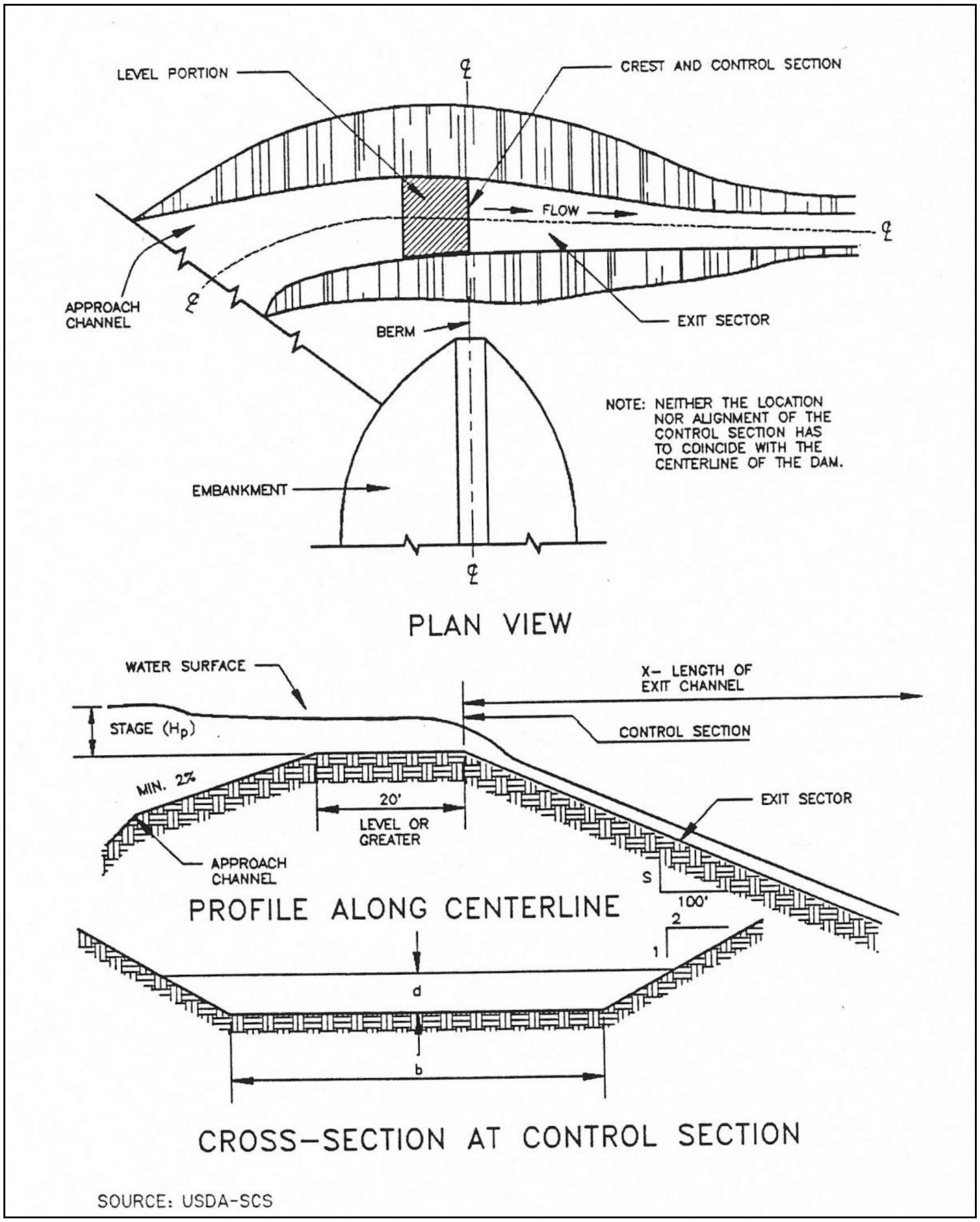


Figure 3.25 Example of Excavated Earth Spillway Design

Table 3.7 Design Data for Earth Spillways

Stage (Hp) In Feet	Spillway Variables	Bottom Width (b) in Feet																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
	S	3.1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	143	150	158
	V	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69

Table 3.7 Design Data for Earth Spillways (continued)

Stage (Hp) In Feet	Spillway Variables	Bottom Width (b) in Feet																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	5	5	5	5	5	5	5	5	5	5	5	5	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6	6	6	6	6	6	6	6	6	6
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	238	253	269	288	301	314	330
	V	5.9	5.9	6	6	6	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	158	175	193	208	226	243	258	275	292	306	323	341	354
	V	6	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	6.2	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	109	109	109	109	109	109	109	109

Source: USDA - SCS

Step 11 Re-estimate the elevation of the design high water and the top of the dam based upon the design of the basin outlet and the emergency spillway.

Step 12 Design the anti-vortex device and trash rack.

If an outfall riser is used, an anti-vortex device and trash rack shall be attached to the top of the basin riser to improve the flow of water into the outfall and prevent floating debris from being carried out of the basin.

This design procedure for the anti-vortex device and trash rack refers only to round riser pipes of corrugated metal. There are numerous ways to provide protection for concrete pipe; these include various hoods and grates and rebar configurations which should be a part of project-specific design and will frequently be a part of a permanent structure.

Refer to Figure 3.26 and Table 3.8. Choose cylinder size, support bars, and top requirements from Table 3.8 based on the diameter of the riser pipe.

Step 13 Design the anchoring for the basin outlet.

The basin outlet must be firmly anchored to prevent its floating.

If the riser is over 10 feet high, the forces acting on the spillway must be calculated. A method of anchoring the spillway which provides a safety factor of 1.25 must be used (downward forces = 1.25 x upward forces).

If the riser is 10 feet or less in height, choose one of the two methods in Figure 3.27 to anchor the basin outlet.

Determine the number and spacing of anti-seep collars for the outfall pipe through the embankment.

Step 14 Provide for dewatering.

(a) Use a modified version of the discharge equation for a vertical orifice and a basic equation for the area of a circular orifice.

Naming the variables:

A = flow area of orifice, in square feet

D = diameter of circular orifice, in inches

h = average driving head (maximum possible head measured from radius of orifice to crest of basin outlet divided by 2), in feet

Q = volumetric flow rate through orifice needed to achieve approximate 6-hour drawdown, cubic feet per second

S = total storage available in dry storage area, cubic feet

Q = S/21,600 seconds

(b) An alternative approach for dewatering is the use of a perforated riser (0.75” to 1” diameter holes spaced every 12 inch horizontally and 8 inch vertically) with 1½ inch to 2 inch filter stone stacked around the exterior.

Use S for basin and find Q. Then substitute in calculated Q and find A:

$$A = \frac{Q}{(0.6) \times \left(64.32 \times \frac{h}{2}\right)} \qquad (3.4)$$

Then, substitute in calculated A and find d:

$$d^* = 2 \times \frac{(A)}{(3.14)} \qquad (3.5)$$

Diameter of the dewatering orifice should never be less than 3 inches in order to help prevent clogging by soil or debris.

Flexible tubing should be at least 2 inches larger in diameter than the calculated orifice to promote improved flow characteristics.

Additional design guidance for orifices and perforated risers are in *Section 2.2.2 of the Hydraulics Technical Manual*.

(c) If a surface skimmer is used as the basin's primary outlet, it may also be used to dewater the basin. Orifice flowrates for the skimmer will be provided by the manufacturer.

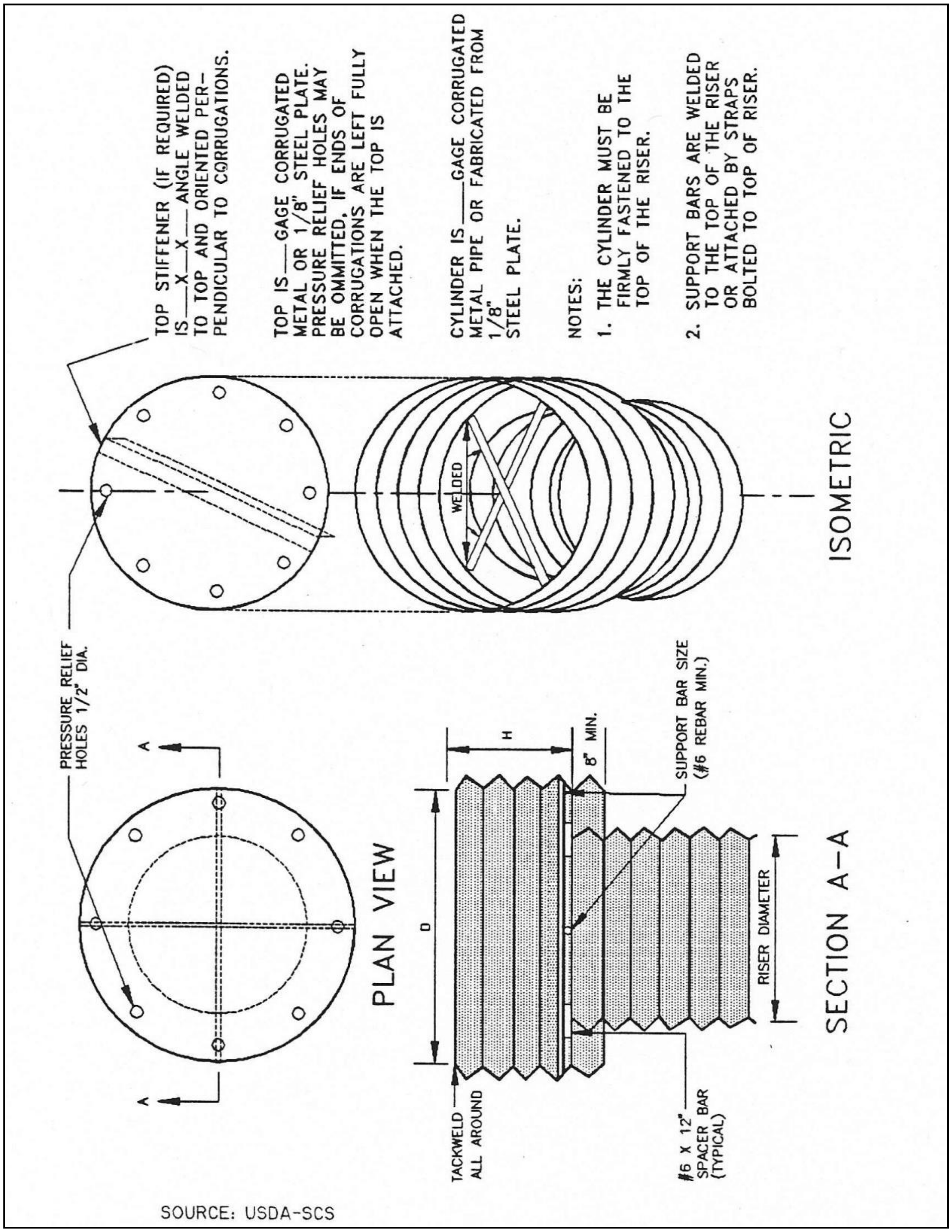


Figure 3.26 Example of Anti-Vortex Design for Corrugated Metal Pipe Riser

Table 3.8 Trash Rack and Anti-Vortex Device Design Table

Riser Diam., in.	Cylinder		Height inches	Minimum Size Support Bar	Minimum Top	
	Diameter inches	Thickness gage			Thickness	Stiffener
12	18	16	6	#6 Rebar or 1 ½ x 1 ½ x 3/16 angle	16 ga. (F&C)	-
15	21	16	7	" "	" "	-
18	27	16	8	" "	" "	-
21	30	16	11	" "	16 ga.(C), 14 ga.(F)	-
24	36	16	13	" "	" "	-
27	42	16	13	" "	" "	-
36	54	14	17	#8 Rebar	14 ga.(C), 12 ga.(F)	-
42	60	16	19	" "	" "	-
48	72	16	21	1 ¼" pipe or 1 ½ x 1 ½ x ¼ angle	14 ga.(C), 10 ga.(F)	-
54	78	16	25	" "	" "	-
60	90	14	29	1 ½" pipe or 1 ½ x 1 ½ x ¼ angle	12 ga.(C), 8 ga.(F)	-
66	96	14	33	2" pipe or 2 x 2 x 3/16 angle	12 ga.(C), 8	2 x 2 x ¼ angle
72	102	14	36	" "	" "	2 ½ x 2 ½ x ¼ angle
78	114	14	39	2 ½" pipe or 2 ½ x ¼ angle	" "	" "
84	120	12	42	2 ½" pipe or 2 ½ x 2 ½ x ¼ angle	" "	2 ½ x 2 ½ x 5/16 angle

Note$_1$: The criterion for sizing the cylinder is that the area between the inside of the cylinder and the outside of the riser is equal to or greater than the area inside the riser. Therefore, the above table is invalid for use with concrete pipe risers.
Note$_2$: Corrugation for 12"-36" pipe measures 2 ⅔ x ½"; for 42"-84" the corrugation measures 5" x 1" or 8" x 1".
Note$_3$: C = corrugated; F = flat.

Source: Adapted from USDA-SCS and Carl M. Henshaw Drainage Products Information.

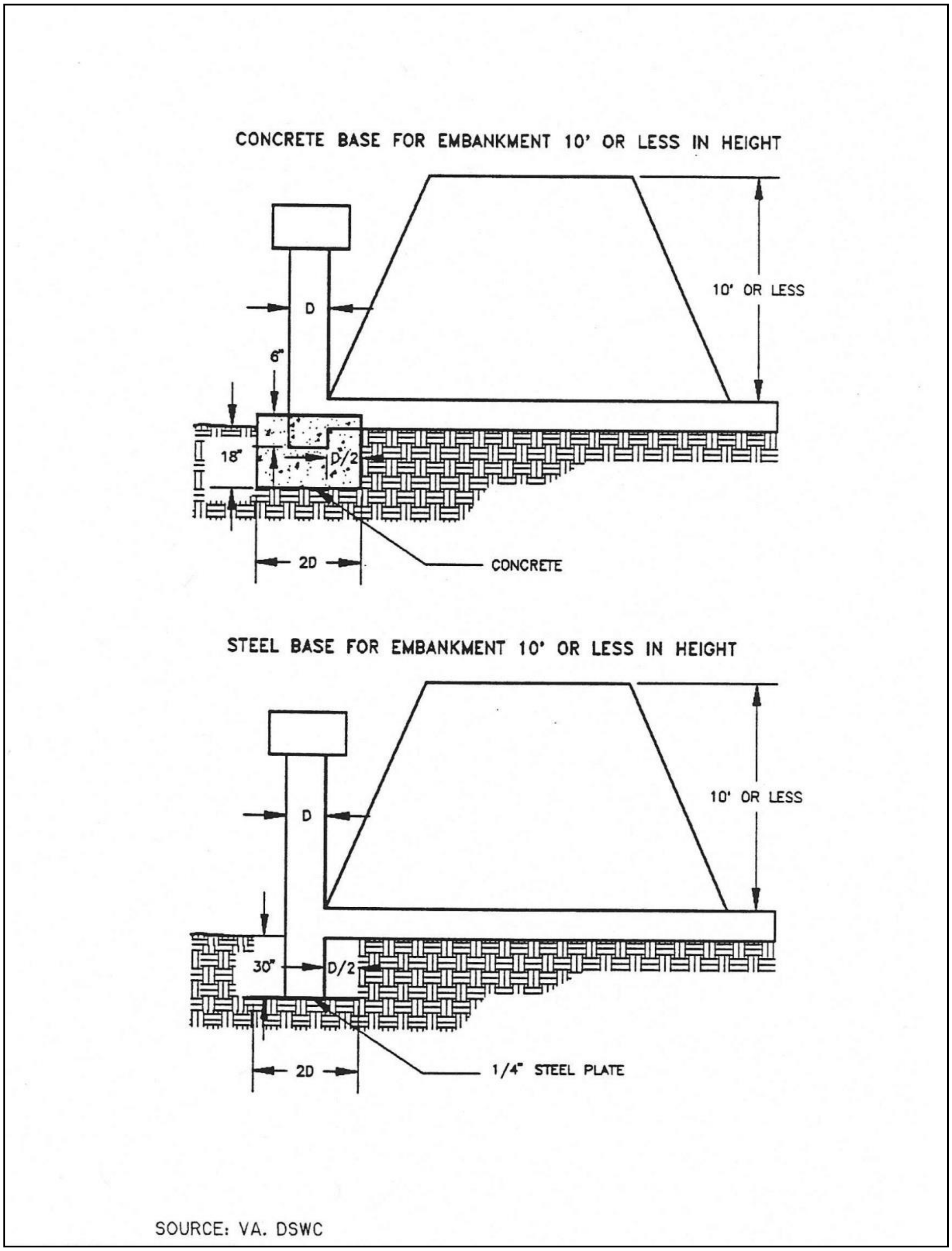


Figure 3.27 Riser Pipe Base Design for Embankment Less Than 10 Feet High

3.9.8 Design Form

Note: This design form is for basins designed with a riser as its primary outlet. It is provided as an example of the type of documentation required for a sediment basin. Different calculations will be needed for other types of outlets.

Project ____

Basin # ______________________ Location _______________________________

Total area draining to basin: ______ acres.

Total disturbed area draining to basin: ______ acres.

Basin Volume Design

1. Minimum required volume is the lesser of

   a.) (3600 cu. ft. x _________ total drainage acres) / 27 = ___________ cu. yds.

   b.) 2 yr, 24 hr storm volume in cubic yards = _____________ cu. yds.

2. Total available basin volume at crest of riser* = ____________ cu. yds. at elevation __________.
   (From Storage - Elevation Curve)

   * Minimum = Lesser of 3600 cubic feet/acre of Total Drainage Area or
   2yr. 24 hr. storm volume from Disturbed Area drained

3. Excavate _________ cu. yds. to obtain required volume*.

   *Elevation corresponding to required volume = invert of the dewatering orifice.

4. Diameter of dewatering orifice = _________________ in.

5. Diameter of flexible tubing = ____________________ in. (diameter of dewatering orifice plus 2 inches).

Preliminary Design Elevations

6. Crest of Riser = ________________________

   Top of Dam = __________________________

   Design High Water = ____________________

   Upstream Toe of Dam = _________________

Basin Shape

7. $\frac{\text{Length of Flow}}{\text{Effective Width}}$ $\frac{L}{We}$ = __________

   If > 2, baffles are not required ____________

   If < 2, baffles are required ______________

Runoff

8. Q_2 = ________________ cfs (From TR-55)

9. Q_{25} = ________________ cfs (From TR-55)

Basin Outlet Design

10. With emergency spillway, required basin outlet capacity $Q_p = Q_2$ = __________ cfs.
    (riser and outfall)

    Without emergency spillway, required basin outlet capacity $Q_p = Q_{25}$ = _______ cfs.
    (riser and outfall)

11. With emergency spillway:

    Assumed available head (h) = __________________ ft. (Using Q_2)

    h = Crest of Emergency Spillway Elevation - Crest of Riser Elevation

    Without emergency spillway:

    h = Design High Water Elevation - Crest of Riser Elevation

12. Riser diameter (D_r) = ______ in. Actual head (h) = __________ft.

    (Figure 3.23)

    Note: Avoid orifice flow conditions.

13. Barrel length (l) = _________ ft.

    Head (H) on outfall through embankment = ________ ft.

    (Figure 3.24)

14. Barrel Diameter = _________ in.

    (From Table 3.5 [concrete pipe] or Table 3.6 [corrugated pipe]).

15. Trash rack and anti-vortex device

    Diameter = _______ inches.

    Height = _________ inches.

    (From Table 3.8).

Emergency Spillway Design

16. Required spillway capacity $Q_e = Q_{25} - Q_p$ = ________ cfs.

17. Bottom width (b) = ________ ft.; the slope of the exit channel(s) = __________ ft./foot; and the minimum length of the exit channel (x) = __________ ft.
    (From Figure 3.25 and Table 3.7).

Final Design Elevations

18. Top of Dam = ________________________

    Design High Water = ___________________

    Emergency Spillway Crest = _____________

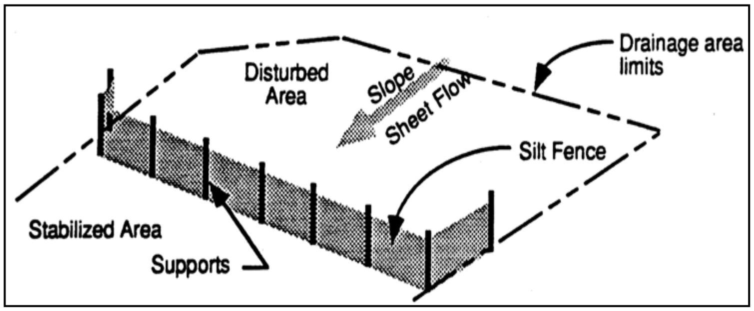
    Basin Riser Crest = ____________________

    Dewatering Orifice Invert = ______________

    Elevation of Upstream Toe of Dam
    (if excavation was performed) = __________

3.10 Silt Fence

Sediment Control



Description: A silt fence consists of geotextile fabric supported by wire mesh netting or other backing stretched between metal posts with the lower edge of the fabric securely embedded six-inches in the soil. The fence is typically located downstream of disturbed areas to intercept runoff in the form of sheet flow. A silt fence provides both filtration and time for sediment settling by reducing the velocity of the runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum drainage area of 0.25 acre per 100 linear feet of silt fence
- Maximum 200 feet distance of flow to silt fence; 50 feet if slope exceeds 10 percent
- Minimum fabric overlap of 3 feet at abutting ends; join fabric to prevent leakage
- Turn end of silt fence line upslope a minimum of 10 feet
- Install stone overflow structure at low points or spaced at approximately 300 feet if no apparent low point

ADVANTAGES / BENEFITS:

- Economical means to treat sheet flow
- Most effective with coarse to silty soil types

DISADVANTAGES / LIMITATIONS:

- Limited effectiveness with clay soils due to clogging
- Localized flooding due to minor ponding at the upslope side of the silt fence
- Not for use as check dams in swales or low areas subject to concentrated flow
- Not for use where soil conditions prevent a minimum toe-in depth of 6 inches or installation of support posts to a depth of 12 inches
- Can fail structurally under heavy storm flows, creating maintenance problems and reducing effectiveness

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Repair undercutting, sags and other fence failures
- Remove sediment before it reaches half the height of the fence
- Repair or replace damaged or clogged filter fabric

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ◒ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control (boxed)
Slope Protection
Sediment Barrier (boxed)
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

Fe=0.50-0.75
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ● Maintenance
- ○ Training
- ◒ Suitability for Slopes > 5%

Other Considerations:

- *Effects of ponding or the redirection of flow onto adjacent areas and property*

3.10.1 Primary Use

Silt fence is normally used as a perimeter control on the down slope side of disturbed areas and on side slopes where stormwater may runoff the area. It is only feasible for non-concentrated, sheet flow conditions. If it becomes necessary to place a silt fence where concentrated flows may be occur (e.g. where two silt fences join at an angle, or across minor channels or gullies), it will be necessary to reinforce the silt fence at that area by a rock berm or sand bag berm, or other structural measures that will support the silt fence.

3.10.2 Applications

Silt fence is an economical means to treat overland, non-concentrated flows for all types of projects. Silt fences are used as perimeter control devices for both site developers and linear (roadway) type projects. They are most effective with coarse to silty soil types. Due to the potential of clogging and limited effectiveness, silt fences should be used with caution in areas that have predominantly clay soil types. In this latter instance, a soils engineer or soil scientist should confirm the suitability of silt fence for that application. Additional controls may be needed to remove fine silts and clay soils suspended in stormwater.

3.10.3 Design Criteria

- Fences are to be constructed along a line of constant elevation (along a contour line) where possible.
- Silt fence can interfere with construction operations; therefore, planning of access routes onto the site is critical.
- Maximum drainage area shall be 0.25 acre per 100 linear feet of silt fence.
- Maximum flow to any 20 foot section of silt fence shall be 1 CFS.
- Maximum distance of flow to silt fence shall be 200 feet or less. If the slope exceeds 10 percent the flow distance shall be less than 50 feet.
- Maximum slope adjacent to the fence shall be 2:1.
- Silt fences shall not be used where there is a concentration of water in a channel, drainage ditch or swale, nor should it be used as a control on a pipe outfall.
- If 50 percent or less soil, by weight, passes the U.S. Standard Sieve No. 200; select the apparent opening size (A.O.S.) to retain 85percent of the soil.
- If 85 percent or more of soil by weight, passes the U.S. Standard Sieve No. 200, silt fences shall not be used unless the soil mass is evaluated and deemed suitable by a soil scientist or geotechnical engineer concerning the erodiblity of the soil mass, dispersive characteristics, and the potential grain-size characteristics of the material that is likely to be eroded.
- Stone overflow structures or other outlet control devices shall be installed at all low points along the fence or spaced at approximately 300 feet if there is no apparent low point.
- Filter stone for overflow structure shall be 1 ½ inches washed stone containing no fines. Angular shaped stone is preferable to rounded shapes.
- Silt fence fabric must meet the following minimum criteria:
    - Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles, 90-lbs.
    - Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 60-lbs.
    - Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 280-psi.

- Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 30(max) to No. 100 (min).
- Ultraviolet Resistance, ASTM D4355 Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture, and Heat in a Xenon Arc Type Apparatus, Minimum 70 percent.

- Fence posts shall be steel and may be T-section or L-section, 1.3 pounds per linear foot minimum, and 4 feet in length minimum. Wood posts may be used depending on anticipated length of service and provided they are 4 feet in length minimum and have a nominal cross section of 2 inches by 4 inches for pine or 2 inches by 2 inches for hardwoods.
- Silt fence shall be supported by steel wire fence fabric as follows:
  - 4 inch x 4 inch mesh size, W1.4 /1.4, minimum 14 gauge wire fence fabric;
  - Hog wire, 12 gauge wire, small openings installed at bottom of silt fence;
  - Standard 2 inch x 2 inch chain link fence fabric; or
  - Other welded or woven steel fabrics consisting of equal or smaller spacing as that listed herein and appropriate gauge wire to provide support.
- Silt Fence shall consist of synthetic fabric supported by wire mesh and steel posts set a minimum of 1-foot depth and spaced not more than 6-feet on center.
- A 6 inch wide trench is to be cut 6 inches deep at the toe of the fence to allow the fabric to be laid below the surface and backfilled with compacted earth or gravel to prevent bypass of runoff under the fence. Fabric shall overlap at abutting ends a minimum of 3 feet and shall be joined such that no leakage or bypass occurs. If soil conditions prevent a minimum toe-in depth of 6 inches or installation of support post to depth of 12 inches, silt fences shall not be used.
- Sufficient room for the operation of sediment removal equipment shall be provided between the silt fence and other obstructions in order to properly maintain the fence.
- The last 10 feet (or more) at the ends of a line of silt fence shall be turned upslope to prevent bypass of stormwater. Additional upslope runs of silt fence may be needed every 200 to 400 linear feet, depending on the traverse slope along the line of silt fence.

3.10.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.5 Silt Fence and in the Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges (TxDot 2004) Item 506.2.J and Item 506.4.C.9.

The American Society for Testing and Materials has established standard specifications for silt fence materials (ASTM D6461) and silt fence installation (ASTM D6462).

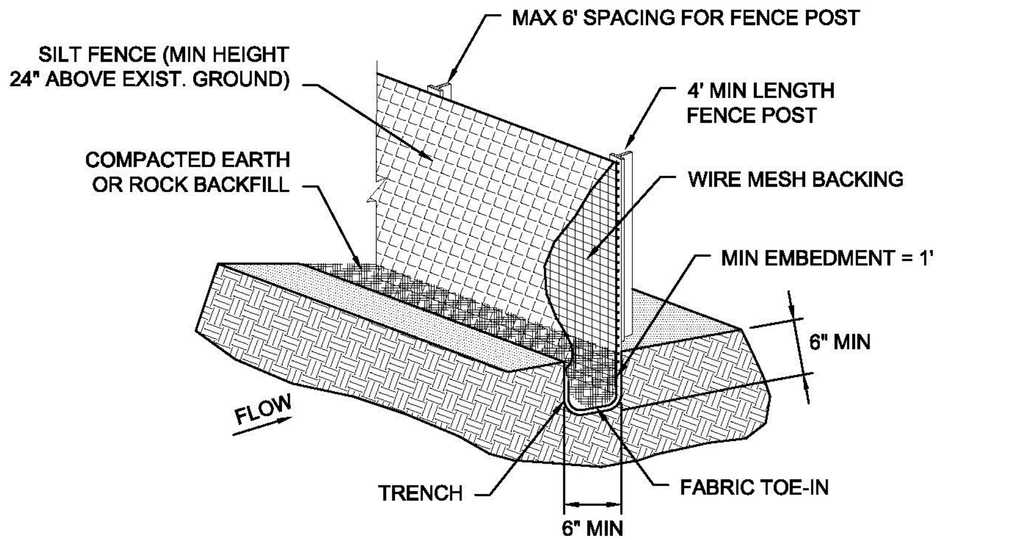
3.10.5 Inspection and Maintenance Requirements

Silt fence should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for buildup of excess sediment, undercutting, sags, and other failures. Sediment should be removed before it reaches half the height of the fence. In addition, determine the source of excess sediment and implement appropriate measures to control the erosion. Damaged or clogged fabric must be repaired or replaced as necessary.

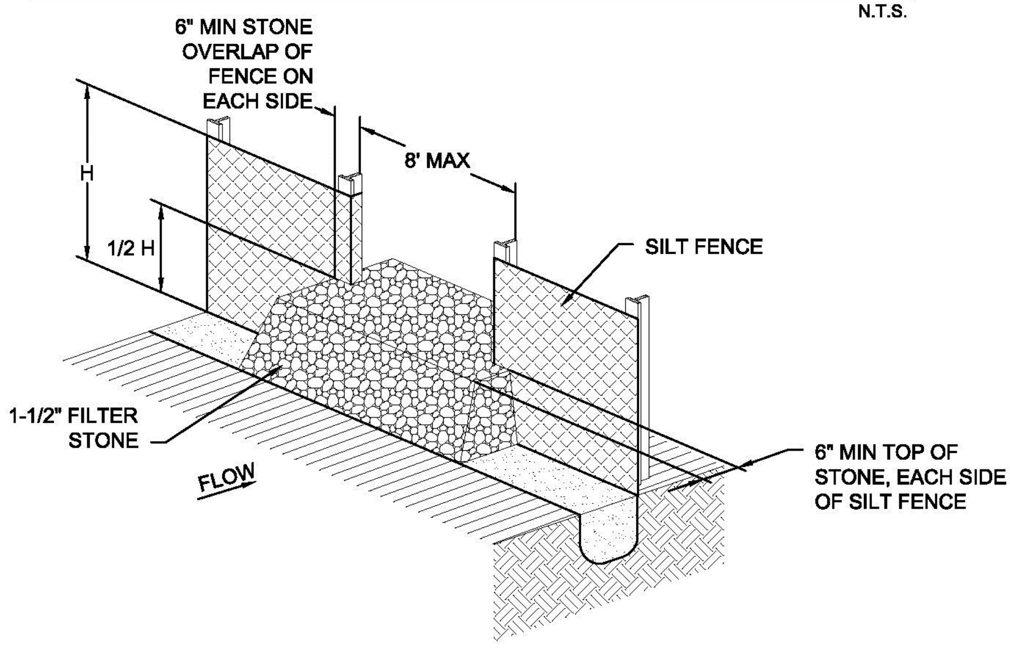
3.10.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



SILT FENCE EXAMPLE
N.T.S.



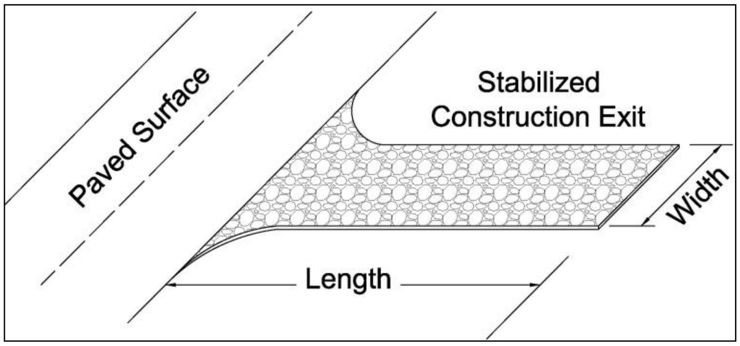
STONE OVERFLOW STRUCTURE EXAMPLE
N.T.S.

NOTES:
1. DESIGN SHALL SHOW ON THE DRAWINGS THE LOCATIONS WHERE OVERFLOW STRUCTURES SHALL BE INSTALLED. OVERFLOW STRUCTURES ARE REQUIRED AT ALL LOW POINTS AND AT A SPACING OF APPROXIMATELY 300 FT WHERE NO LOW POINT IS APPARENT.
2. DESIGNER SHALL ON THE DRAWINGS THE LOCATIONS WHERE SILT FENCE IS TO BE TURNED UPSLOPE. UPSLOPE LENGTHS SHALL BE A MINIMUM OF 10 FEET.

Figure 3.28 Schematics of Silt Fence

3.11 Stabilized Construction Exit

Sediment Control



Description: A stabilized construction exit is a pad of crushed stone, recycled concrete or other rock material placed on geotextile filter cloth to dislodge soil and other debris from construction equipment and vehicle tires prior to exiting the construction site. The object is to minimize the tracking of soil onto public roadways where it will be suspended by stormwater runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Slope exit away from offsite paved surface
- Minimum width and length dependent on size of disturbed area, which correlates to traffic volume
- 6 inches minimum thickness of stone layer
- Stone of 3 to 5 inches in size
- Add a wheel cleaning system when inspections reveal the stabilized exit does not prevent tracking

ADVANTAGES / BENEFITS:

- Reduces tracking of soil onto public streets
- Directs traffic to a controlled access point
- Protects other sediment controls by limiting the area disturbed

DISADVANTAGES / LIMITATIONS:

- Effectiveness dependent on limiting ingress and egress to the stabilized exit
- A wheel washing system may also be required to remove clay soil from tires, particularly in wet conditions

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Replace rock when sediment in the void area between the rocks is visible on the surface
- Periodically re-grade and top dress with additional stone to maintain efficiency

TARGETED POLLUTANTS

◒ Sediment
○ Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
$\boxed{\textbf{Temporary Stabilization}}$
Final Stabilization
Waste Management
Housekeeping Practices

Fe=N/A

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
○ Training
○ Suitability for Slopes > 5%

Other Considerations:

- *None*

3.11.1 Primary Use

Stabilized construction exits are used to remove soil, mud and other matter from vehicles that drive off of a construction site onto public streets. Stabilized exits reduce the need to remove sediment from streets. When used properly, they also control traffic by directing vehicles a single (or two for larger sites) location. Controlling traffic onto and off of the site reduces the number and quantity of disturbed areas and provides protection for other sediment controls by decreasing the potential for vehicles to drive over the control.

3.11.2 Applications

Stabilized construction exits are used on all construction sites with a disturbed area of one acre or larger and are a recommended practice for smaller construction sites. A stabilized exit is used on individual residential lots until the driveway is placed. Stabilized construction exits may be used in conjunction with wheel cleaning systems as described in *Section 3.16 Wheel Cleaning Systems*.

3.11.3 Design Criteria

- Limit site access to one route during construction, if possible; two routes for linear and larger projects.
- Prevent traffic from avoiding or shortcutting the full length of the construction exit by installing barriers. Barriers may consist of silt fence, construction safety fencing, or similar barriers.
- Design the access point(s) to be at the upslope side of the construction site. Do not place construction access at the lowest point on the construction site.
- Stabilized construction exits are to be constructed such that drainage across the exit is directed to a controlled, stabilized outlet onsite with provisions for storage, proper filtration, and removal of wash water.
- The exit must be sloped away from the paved surface so that stormwater from the site does not discharge through the exit onto roadways.
- Minimum width of exit shall be 15 feet.
- The construction exit material shall be a minimum thickness of 6 inches. The stone or recycled concrete used shall be 3 to 5 inches in size with little or no fines.
- The geotextile fabric must meet the following minimum criteria:
    - Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles, 300 lbs.
    - Puncture Strength, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 120 lbs.
    - Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 600 psi.
    - Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 40 (max).
- Rock by itself may not be sufficient to remove clay soils from wheels, particularly in wet conditions. When necessary, vehicles must be cleaned to remove sediment prior to entering paved roads, streets, or parking lots. Refer to *Section 3.16 Wheel Cleaning Systems* for additional controls.
- Using water to wash sediment from streets is prohibited
- Minimum dimensions for the stabilized exit shall be as follows:

Table 3.9 Minimum Exit Dimensions		
Disturbed Area	***Min. Width of Exit***	***Min. Length of Exit***
< 1 Acre	15 feet	20 feet
$\geq$ 1 Acre but < 5 Acres	25 feet	50 feet
$\geq$ 5 Acres	30 feet	50 feet

- If a wheel cleaning system is used, the width of the stabilized exit may be reduced to funnel traffic into the system. Refer to *Section 3.16 Wheel Cleaning*.

3.11.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.10 Stabilized Construction Entrance and in the Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges (TxDOT 2004) Item 506.2.E and Item 506.4.C.5.

3.11.5 Inspection and Maintenance Requirements

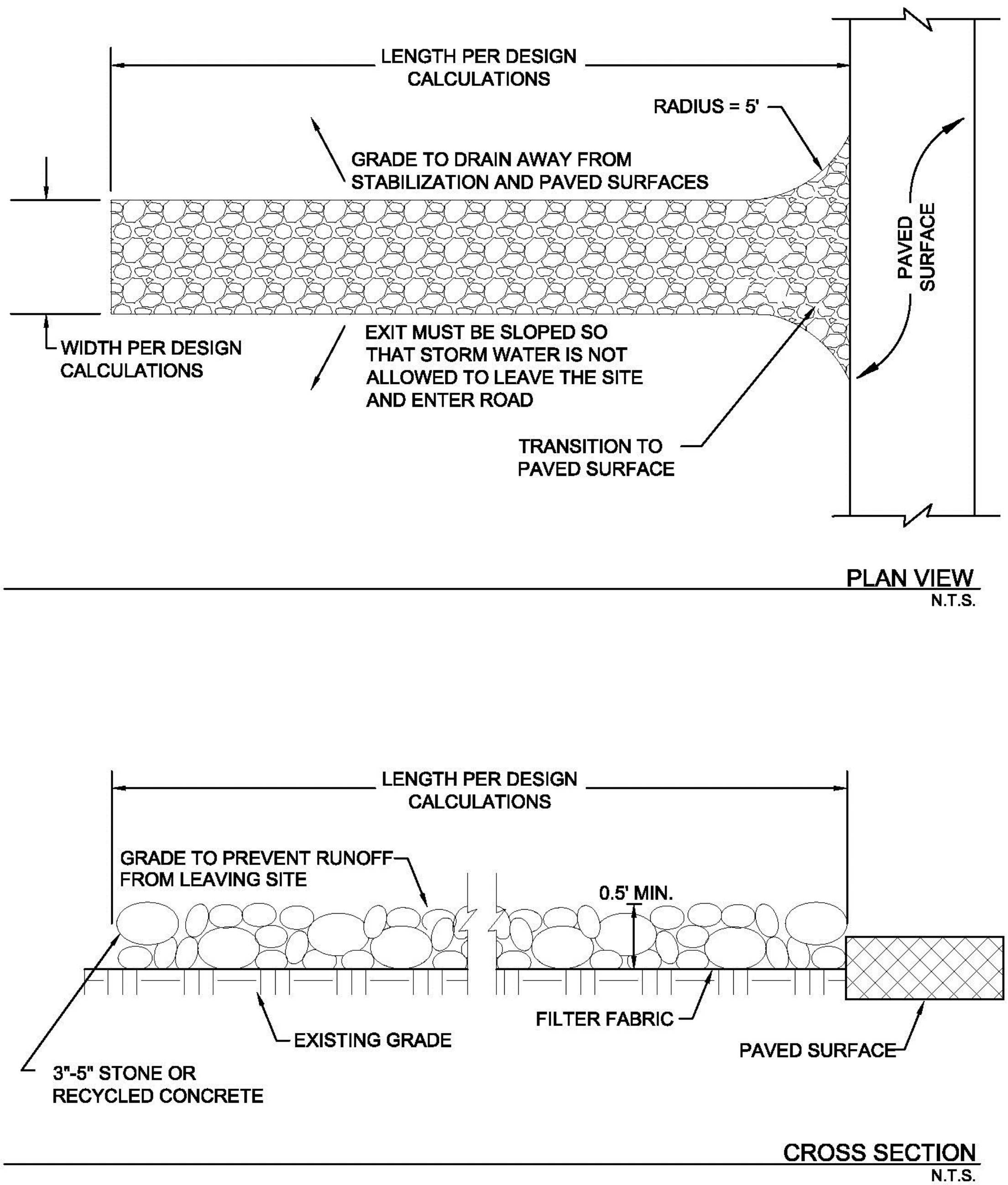
Construction exits should be inspected regularly (at least as often as required by the TPDES Construction General Permit). The stabilized construction exit shall be maintained in a condition that prevents tracking or flow of sediment onto paved surfaces. Periodic re-grading and top dressing with additional stone must be done to keep the efficiency of the exit from diminishing. The rock shall be re-graded when ruts appear. Additional rock shall be added when soil is showing through the rock surface.

Additional controls are needed if inspections reveal a properly installed and maintained exit, but tracking of soil outside the construction area is still evident. Additional controls may be daily sweeping of all soil spilled, dropped, or tracked onto public rights-of-way or the installation of a wheel cleaning system.

3.11.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

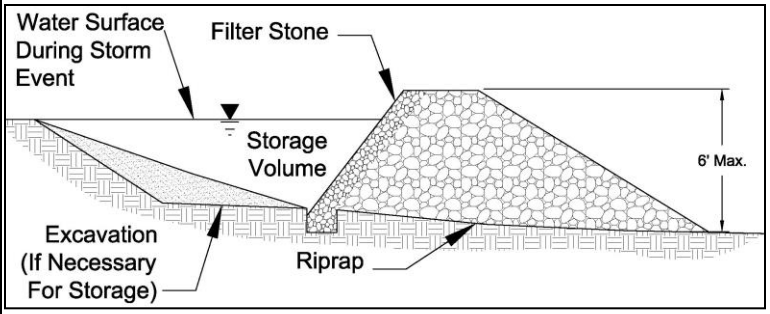


NOTE: INSTALL SILT FENCE, CONSTRUCTION SAFETY FENCING, OR SIMILAR BARRIER ALONG THE EXIT TO DIRECT TRAFFIC INTO THE EXIT.

Figure 3.29 Schematics of Stabilized Construction Exit

3.12 Stone Outlet Sediment Trap

Sediment Control



Description: A stone outlet sediment trap is a small detention area formed by placing a stone embankment with an integral stone filter outlet across a drainage swale for the purpose of detaining sediment-laden runoff from construction activities. The sediment trap detains runoff long enough to allow most of the suspended sediment to settle while still allowing for diffused flow of runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum contributing drainage area of 10 acres for excavated trap and 5 acres for bermed trap
- Provide storage volume for the 2-year, 24-hour design storm
- Maximum embankment height of 6 feet
- Embankment slope of 1.5:1 or flatter
- 2 foot minimum top width

ADVANTAGES / BENEFITS:

- Effectively traps sediment in a drainage swale
- Reduces flow velocities
- Relatively long effective life

DISADVANTAGES / LIMITATIONS:

- Amount of land required
- Can cause minor upstream flooding, possibly impacting construction operations
- Not for use in “live” (normally flowing) channels

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Replace filter stone when it appears to be silted in such that efficiency is diminished
- Remove trash and debris after each storm event
- Remove deposited sediment when before the storage capacity is reduced by one third or has reached a depth of one foot, whichever is less

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ◒ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.50-0.85

(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ◒ Maintenance
- ○ Training
- ● Suitability for Slopes > 5%

Other Considerations:

- *Re-grading and stabilization of the control area after construction*

3.12.1 Primary Use

A sediment trap is used where flows are concentrated in a drainage swale or channel. The sediment trap detains and temporarily impounds stormwater, which allows for settling of sediment as the water is slowly discharged from the trap. Sediment traps may be used in combination with check dams when erosive velocities exist in the swale upstream of the sediment trap.

3.12.2 Applications

Temporary stone outlet sediment traps are installed at locations where concentrated flows require a protected outlet to contain sediment or spread flow prior to discharge. They are an effective, long term (12 – 18 months) application for sediment control on large construction sites where a sediment basin is not feasible due to site or construction method restrictions. Several traps may be used to control sediment on drainage sub-basins within the construction site, instead of one large sediment basin at the discharge point from the entire construction site. Sediment traps may also be used with a passive treatment system to provide better removal of fine silt and clay soil particles.

3.12.3 Design Criteria

- Design calculations are required for the use of this control. The designer shall provide drainage computations and dimensions for the stone outlet, berms, and excavated areas associated with this control.
- The maximum drainage area contributing to the trap shall be less than 10 acres for the excavated stone outlet sediment trap and 5 acres or less for the bermed trap.
- The minimum storage volume shall be the volume of runoff from the temporary control design storm (2-year, 24 hour) for the sediment trap's drainage area.
- The surface area of the design storage area shall not be less than 1 percent of the area draining to the device.
- The maximum height of the rock shall be 6 feet, as measured from the toe of the slope on the downstream side to the low point in the rock dam.
- Minimum width of the rock dam at the top shall be 2 feet.
- Rock dam slope shall be 1.5:1 or flatter.
- The rock dam shall have a depressed area, over the center of swale, to serve as the outlet with a minimum width of 4 feet.
- A six inch minimum thickness layer of 1½ inch filter stone shall be placed on the upstream face of the stone embankment when the stormwater runoff contains fine silt and clay soil particles.
- The embankment shall be comprised of well graded stone with a size range of 6 to 12 inches in diameter. The stone may be enclosed in wire mesh or gabion basket and anchored to the channel bottom to prevent washing away.
- The dam shall consist of stone riprap or a combination of compacted fill with a stone riprap outlet.
- Fill placed to constrict the swale for construction of the excavated stone outlet sediment trap and fill placed for the berm in the bermed stone outlet sediment trap shall consist of clay material, minimum Plasticity Index of 30, using ASTM D4318 Standard Test for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
- Fill shall be placed in 8 inch loose lifts (maximum) and compacted to 95% Standard Proctor Density at optimum moisture content using ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort.
- The outlet shall be designed to have a minimum freeboard of 6" at design flow.

- Rock shall be placed on geotextilefilter fabric meeting the following minimum criteria:
  - Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles, 250-lbs.
  - Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 135-lbs.
  - Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 420-psi.
  - Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 20 (max).
- The geotextile fabric, covered with a layer of stone, shall extend past the base of the embankment on the downstream side a minimum of 2 feet.

3.12.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.12 Stone Outlet Sediment Trap.

3.12.5 Inspection and Maintenance Requirements

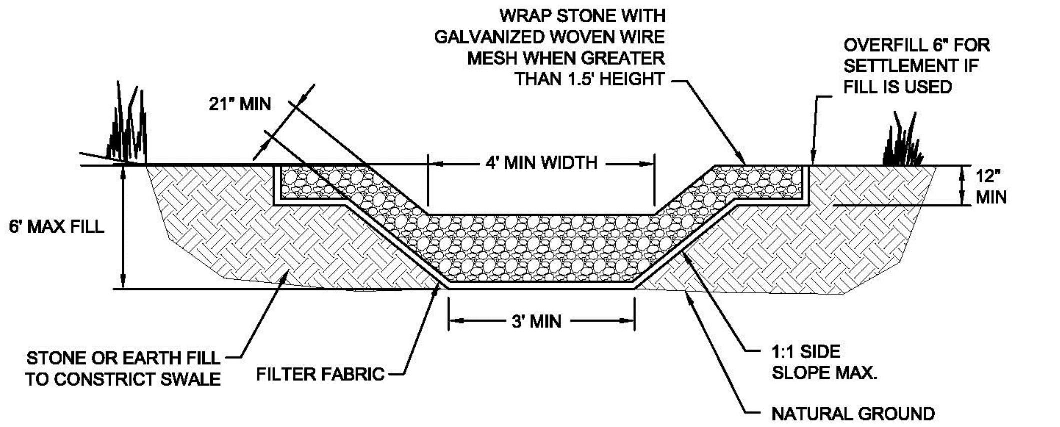
The stone outlet sediment trap should be inspected regularly (at least as often as required by the TPDES Construction General Permit) to check for clogging of the void spaces between stones. If the filter stone appears to be clogged, such that the basin will not completely drain, then the filter stone will require maintenance. If the filter stone is not completely clogged it may be raked with a garden rake to allow the water to release from the basin. If filter stone is completely clogged with mud and sediment, then the filter stone will have to be removed and replaced. Failure to keep the filter stone material properly maintained will lead to clogging of the stone riprap embankment. When this occurs, the entire stone rip-rap structure will need to be replaced. If the aggregate appears to be silted in such that efficiency is diminished, the stone should be replaced.

Trash and debris should be removed from the trap after each storm event to prevent it from plugging the rock. Deposited sediment shall be removed before the storage capacity is decreased by one-third, or sediment has reached a depth of one foot, whichever is less. The removed sediment shall be stockpiled or redistributed in areas that are protected with erosion and sediment controls.

3.12.6 Example Schematics

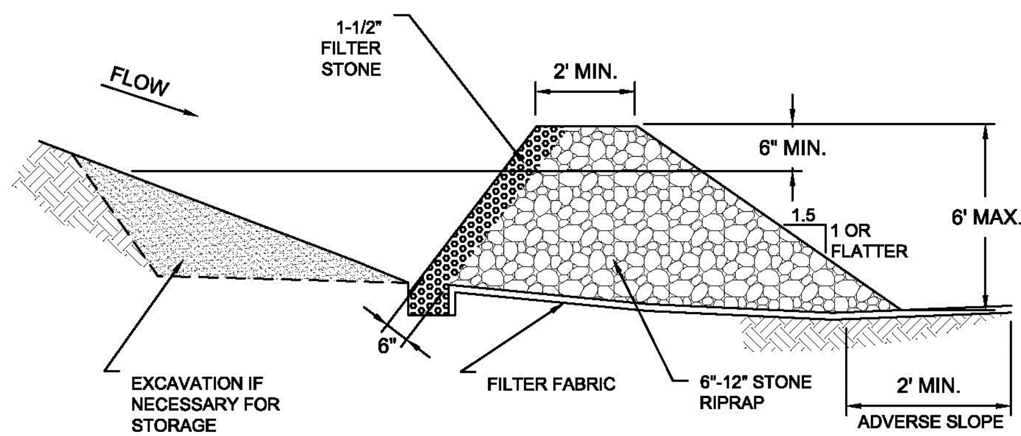
The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



EXCAVATED STONE OUTLET SEDIMENT TRAP VIEW LOOKING UPSTREAM

N.T.S.



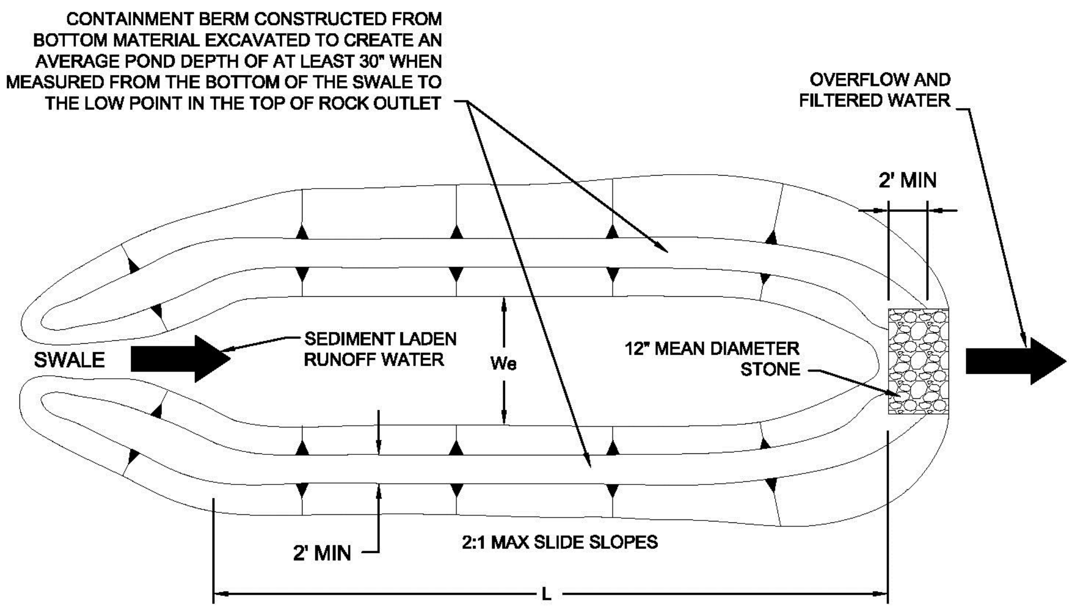
EXCAVATED STONE OUTLET SEDIMENT TRAP SECTION VIEW

N.T.S.

NOTE: ACTUAL DIMENSIONS OF THE SEDIMENT TRAP SHALL BE DESIGNED BASED ON FLOW CONDITIONS AND SITE TOPOGRAPHY. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETER USED TO DESIGN THE TRAP.

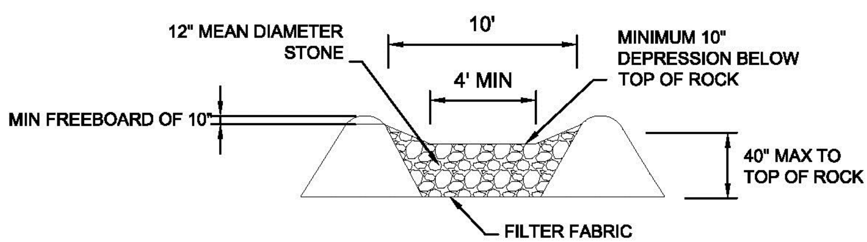
- SIZE OF CONTRIBUTING DRAINAGE AREA
- DESIGN STORM VOLUME AND FLOW RATE AT THE TRAP
- HEIGHT, SLOPE, AND LENGTH OF STONE OUTLET
- STORAGE VOLUME
- EXTENT OF GRADING TO PROVIDE THE CONTROLLED OUTLET

Figure 3.30 Schematics of Excavated Stone Outlet Sediment Trap



BERMED STONE OUTLET SEDIMENT TRAP PLAN VIEW

N.T.S.



BERMED STONE OUTLET SEDIMENT TRAP SECTION VIEW

N.T.S.

NOTE: ACTUAL DIMENSIONS OF THE SEDIMENT TRAP SHALL BE DESIGNED BASED ON FLOW CONDITIONS AND SITE TOPOGRAPHY. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETER USED TO DESIGN THE TRAP.

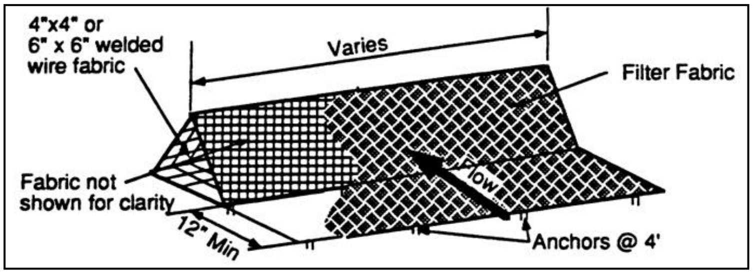
- SIZE OF CONTRIBUTING DRAINAGE AREA
- DESIGN STORM VOLUME AND FLOW RATE AT THE TRAP
- HEIGHT, SLOPE, AND LENGTH OF STONE OUTLET
- STORAGE VOLUME
- EXTENT OF GRADING TO PROVIDE THE CONTROLLED OUTLET

TRIBUTARY AREA (ACRES)	L (FT)	We (FT)
< 0.5	59	13
0.51-1.0	82	16
1.01-1.5	102	20
1.51-2.0	118	23
2.01-2.5	131	26
2.51-3.0	144	30
3.01-3.5	154	30
3.51-4.0	167	33
4.01-4.5	177	36
4.51-5.0	187	36

Figure 3.31 Schematics of Bermed Stone Outlet Sediment Trap
(Source: City of Chesterfield Department of Public Works Detail SC 7.2)

3.13 Triangular Sediment Filter Dike

Sediment Control



Description: A triangular sediment filter dike is a self-contained silt fence consisting of filter fabric wrapped around welded wire fabric and shaped into a triangular cross section. While similar in use to a silt fence, the dike is reusable, sturdier, transportable, and can be used on paved areas or in situations where it is impractical to install embedded posts for support.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum drainage area of 0.25 acre per 100 linear feet of dike
- Maximum 200 feet distance of flow to filter dike; 50 feet if slope exceeds 10 percent
- Overlap ends of filter material 6 inches to cover dike-to-dike junction; secure with shoat rings

ADVANTAGES / BENEFITS:

- Can be installed on paved surfaces or where the soil type prevents embedment of other controls
- Withstands more concentrated flow and higher flow rates than silt fence

DISADVANTAGES / LIMITATIONS:

- Localized flooding due to minor ponding at the upslope side of the filter dike
- Not effective where there are substantial concentrated flows
- Not effective along contours due to the potential for flow concentration and overtopping

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove sediment before it reaches 6 inches in depth
- Clean or replace fabric if clogged
- Repair or replace dike when structural deficiencies are found

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ◒ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

- $\boxed{\textbf{Perimeter Control}}$
- **Slope Protection**
- $\boxed{\textbf{Sediment Barrier}}$
- **Channel Protection**
- **Temporary Stabilization**
- **Final Stabilization**
- **Waste Management**
- **Housekeeping Practices**

Fe=0.50-0.75
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ◒ Maintenance
- ○ Training
- ◒ Suitability for Slopes > 5%

Other Considerations:

- *Effects of ponding on adjacent areas and property*

3.13.1 Primary Use

Triangular filter dikes are used in place of silt fence, treating sediment flow at the perimeter of construction areas and at the perimeter of the site. Also, the dikes can serve as stream protection devices by preventing sediment from entering the streams or as check dams in small swales.

Triangular sediment filter dikes are especially useful for construction areas surrounded by pavement, where silt fence, filter berm, or other sediment control installations are impractical.

3.13.2 Applications

Triangular dikes are used to provide perimeter control by detaining sediment on a disturbed site with drainage that would otherwise flow onto adjacent properties. Triangular dikes function as sediment trapping devices when used in areas of sheet flow across disturbed areas or are placed along stream banks to prevent sediment-laden sheet flow from entering the stream. The dikes can be subjected to more concentrated flows and a higher flow rate than silt fence.

Dikes can be used on a variety of surfaces where other controls are not effective. They may be installed on paved surfaces and where the soil type prevents embedment of other sediment controls.

3.13.3 Design Criteria

- Dikes are to be installed along a line of constant elevation (along a contour line).
- Maximum drainage area shall be 0.25 acre per 100 linear feet of dike.
- Maximum flow to any 20 foot section of dike shall be 1 CFS.
- Maximum distance of flow to dike shall be 200 feet or less. If the slope exceeds 10 percent, the flow distance shall be less than 50 feet.
- Maximum slope adjacent to the dike shall be 2:1.
- If 50 percent or less of soil, by weight, passes the U.S. Standard Sieve No. 200, select the apparent opening size (A.O.S.) to retain 85 percent of the soil.
- If 85 percent or more of soil, by weight, passes the U.S. Standard Sieve No. 200, triangular sediment dike shall not be used due to clogging.
- The filter fabric shall meet the following minimum criteria:
    - Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles 90-lbs.
    - Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 60-lbs.
    - Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 280-psi.
    - Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Siev No. 30 (max) to 100 (min).
    - Ultraviolet Resistance, ASTM D4355 Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture, and Heat in a Xenon Arc Type Apparatus, Minimum 70 percent.
- The internal support for the dike structure shall be 6-gauge 6 inch x 6 inch wire mesh or 6-guage 4 inch x 4 inch welded wire fabric folded into triangular form eighteen (18) inches on each side.
- Tie-in to the existing grade should be accomplished by:

  (i) embedding the fabric six-inches below the top of ground on the upslope side;

(ii) extending the fabric to form a 12 inch skirt on the upstream slope and covering it with 3 to 5 inches of 1½ inch washed filter stone; or

(iii) entrenching the base of the triangular dike four inches below ground.

For (ii) above, the skirt and the upslope portion of the triangular dike skeleton should be anchored by metal staples on two-foot centers, driven a minimum of six inches into the ground (except where crossing pavement or exposed limestone). When installed on pavement, the washed rock in option (ii) may be replaced by bags filled with 1½ inch washed filter stone placed at 4 foot spacing to anchor the end of the filter fabric to the pavement.

- Filter material shall lap over ends six (6) inches to cover dike-to-dike junction; each junction shall be secured by shoat rings. Where the dike is placed on pavement, two rock bags shall be used to anchor the overlap to the pavement. Additional bags shall be used as needed to ensure continuous contact with the pavement (no gaps).
- Sand bags or large rock should be used as ballast inside the triangular dike section to stabilize the dike against the effects of high flows.
- Sufficient room for the operation of sediment removal equipment shall be provided between the dike and other obstructions in order to properly remove sediment.
- The ends of the dike shall be turned upgrade to prevent bypass of stormwater.
- When used as a perimeter control on drainage areas larger than 0.5 acres, a stone overflow structure, similar to the one shown in *Section 3.10 Silt Fence*, may be necessary at low points to act as a controlled overflow point in order to prevent localized flooding and failure of the dike.
- If used as check dams in small swales (drainage areas less than 3 acres), the dikes shall be installed according to the spacing and other criteria in *Section 2.1 Check Dam*.

3.13.4 Design Guidance and Specifications

Specifications for construction of this item may be found in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments, Section 201.8 Triangular Sediment Filter Dike.

3.13.5 Inspection and Maintenance Requirements

Triangular sediment filter dikes should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Sediment should be removed before it reaches 6 inches in depth. If the fabric becomes clogged, it should be cleaned or, if necessary, replaced. If structural deficiencies are found, the dike should be immediately repaired or replaced.

The integrity of the filter fabric is important to the effectiveness of the dike. Overlap between dike sections must be checked on a regular basis and repaired if deficient.

3.13.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

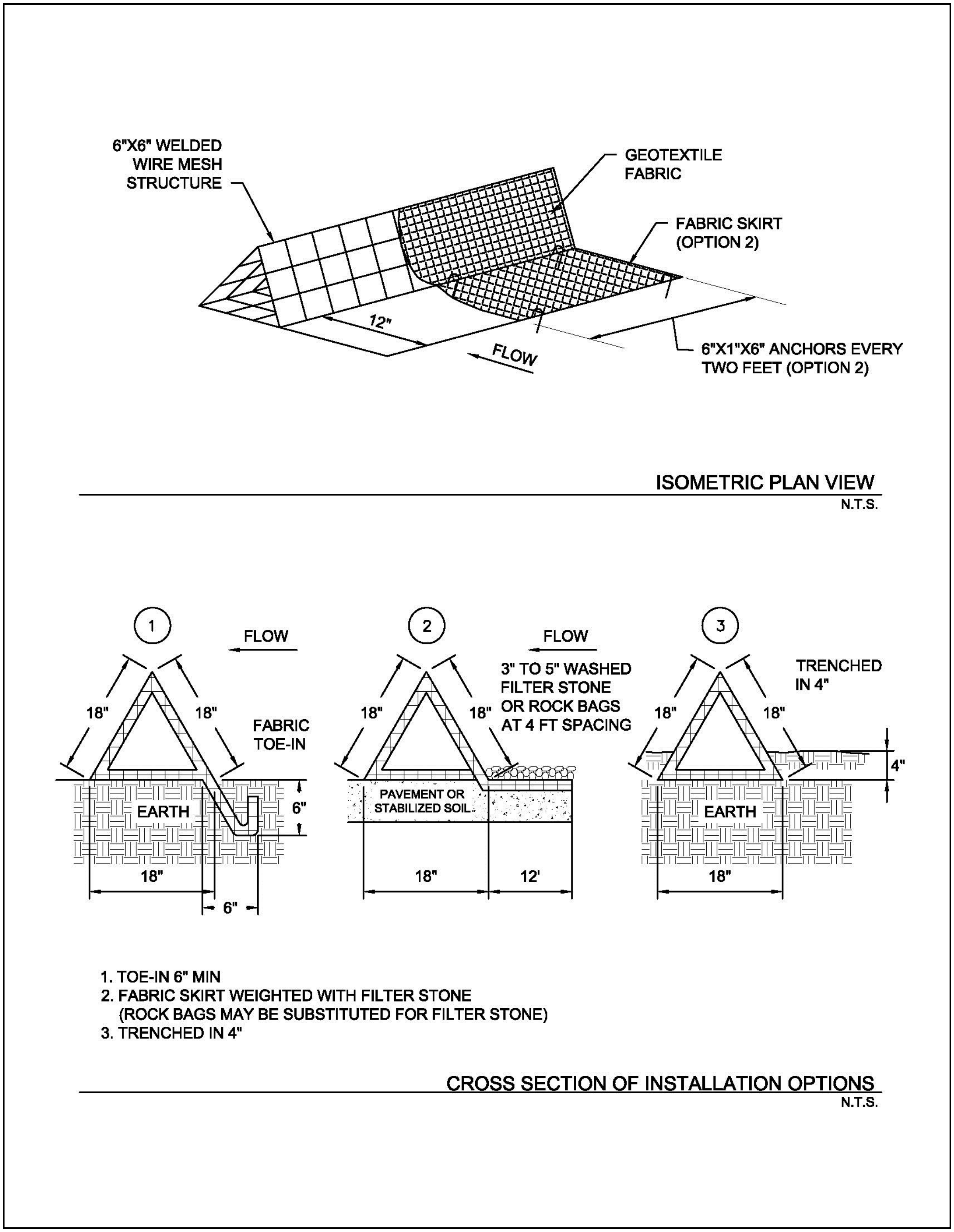
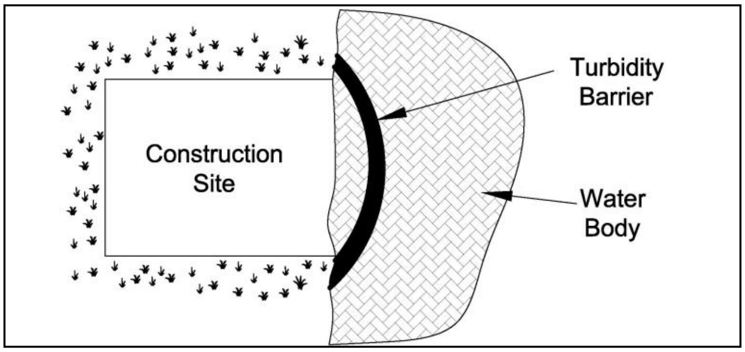


Figure 3.32 Schematics of Triangular Sediment Filter Dike

3.14 Turbidity Barrier

Sediment Control



Description: A turbidity barrier is a floating geotextile or PVC curtain that is designed to control sediment within a body of water. It is also known as a floating silt barrier or turbidity/silt curtain. The barrier typically consists of floats, curtain, ballast, and anchor lines. The barrier may be permeable or impermeable. Barriers of 100 feet or longer are constructed of a series of connected panels.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Barrier specified based on depths and velocities in the water body in which the barrier is installed
- Installation and anchoring according to manufacturer's recommendations
- Height of barrier 10 percent greater than design water depth
- Specified length of barrier 10 to 20 percent greater than design length

ADVANTAGES / BENEFITS:

- Controls sediment from construction activities where other types of down slope barriers are infeasible
- Protects sensitive wetlands and water bodies
- May be re-used on different projects

DISADVANTAGES / LIMITATIONS:

- Limited usefulness in water bodies with high velocities
- May be damaged by a large storm event
- Barrier can be difficult to remove when under heavy sediment accumulations

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Repair or replace fabric as needed
- Re-anchor if dislodged

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ● Oil & Grease
- ● Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control (boxed)

Slope Protection

Sediment Barrier (boxed)

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.50-0.90
(*Depends on soil type*)

IMPLEMENTATION CONSIDERATIONS

- ● Capital Costs
- ◒ Maintenance
- ○ Training
- ● Suitability for Slopes > 5%

Other Considerations:

- *Conflicts with boat traffic*

3.14.1 Primary Use

Turbidity barriers are used when construction activities will disturb the bank of a perennial stream, river, pond, or lake. They are also used when construction activities require construction of a coffer dam, low water crossing, or other activity that will disturb soil within a water body.

3.14.2 Applications

Turbidity barriers are used on development projects that have a perennial water body within or adjacent to the development. The barrier floats in the water and is anchored at the bottom and/or sides depending on the site conditions. Where construction activities extend down a bank of the water body into the water surface, it is installed along the length of disturbed area and functions as a down slope perimeter control.

The barriers are also used where linear projects cross a water body, development extends into a water body, or temporary coffer dams are installed to facilitate construction. In these applications, the turbidity barrier functions as a sediment trap for soil suspended in the water body by construction activities.

Turbidity barriers are most applicable where special aquatic sites or sensitive receiving waters need to be protected. Examples of these types of waters included wetlands regulated under Section 404 of the Clean Water Act, spring-fed water bodies, water bodies with a Total Maximum Daily Load, construction sites with an effluent limit, and water bodies with species protected under the Federal Endangered Species Act or the State of Texas Threatened and Endangered Species Regulations.

3.14.3 Design Criteria

- Specific design information is required for the use of this control. The designer shall specify the manufacturer, type of turbidity barrier, length, and anchoring mechanism based on the site conditions, range of depths and velocities in the water body, and project duration.
- The type of turbidity barrier must be specified in accordance with the manufacturer's guidance for the depth of water, salinity, velocities, wave height, and project duration.
- If the barrier will be used to contain contaminants in addition to sediment, ensure the barrier's material is compatible with the contaminant of concern.
- Fabrics used to construct the curtain shall be woven and coated for UV protection.
- Fabric minimum grab tensile strength shall be 202 pounds using ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles for velocities of 0.5 feet per second or less. Higher velocities require an engineer's design, typically provided by the manufacturer.
- The height of the barrier shall be 10 percent greater than the design water depth to ensure the bottom of the barrier rests on the ground.
- The physical length of the barrier as purchased from the manufacturer shall be 10 to 20 percent longer than the design length to reduce stress on the barrier and make installation easier.
- Panel lengths shall be a maximum of 100 feet in water less than 13 feet and 50 feet in water of 13 feet or deeper.
- Minimize the area to be enclosed by the barrier.
- Provide a means to remove captured trash and sediment from behind the turbidity barrier before the barrier is removed, unless the potential for re-suspending the sediment is greater than the benefit of removing it. Removed sediment will be saturated with water. If possible, reserve a space onsite for the sediment to be spread for drying. Otherwise, provide water-tight containers and disposal procedures for the wet sediment.
- Sediment-laden water may be removed from behind the barrier using dewatering procedures discussed in *Section 3.3 Dewatering Controls*.

- Barriers shall be designed at a slant to the direction of flow to decrease pressure on the curtain. Barriers should not be installed perpendicular to flow.
- On large lakes where reversing currents may exist, design the barrier to be anchored on both sides of the curtain.
- On lakes or other bodies of water that may have boat traffic, install a buoy marker on any anchors or anchor lines that extend into the water beyond the visible surface of the turbidity barrier.

3.14.4 Design Guidance and Specifications

No specification for construction of turbidity barriers is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.14.5 Inspection and Maintenance Requirements

The turbidity barrier should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for movement or dislodgement of the barrier. Verify that all floats are intact and that anchors are secure. The entire top edge should be visible above the water surface. Re-anchor or re-enforce the anchors if the barrier has moved.

Check for debris that may have floated into the barrier and damaged it. Also look for and remove debris caught in the fabric or sediment collected in pockets of the fabric. The fabric should be free of tears and gaps. Repair and replace fabric where damage has occurred.

Ensure panel connections are secure and in good condition. Repair any tears in the fabric at the connection points.

Remove sediment from folds and pleats in the barrier when there is evidence of the barrier being pulled down by the weight of the sediment. All sediment accumulated behind the barrier shall be removed from the water before the barrier is removed.

3.14.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

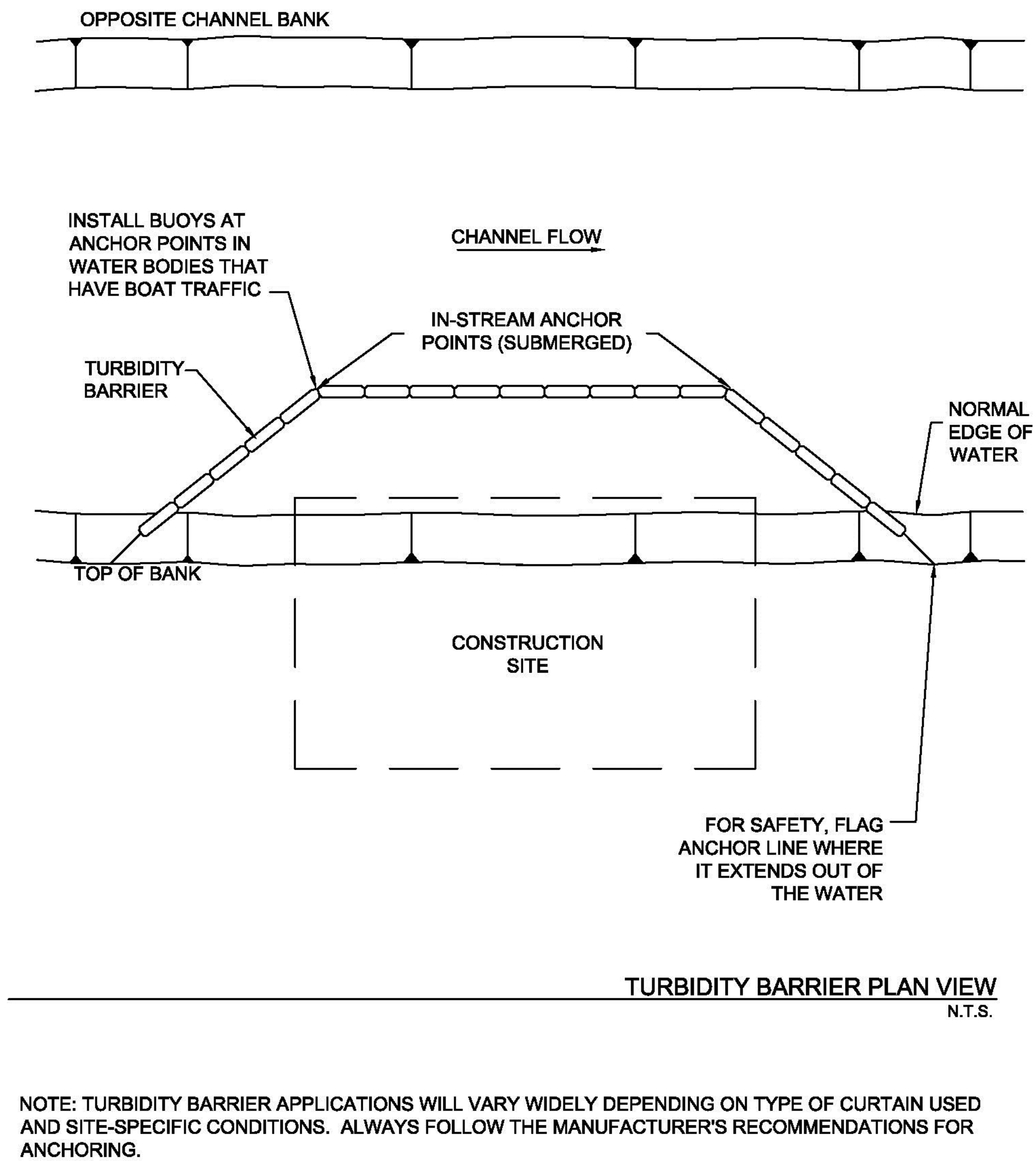


Figure 3.33 Example Application of Turbidity Barrier

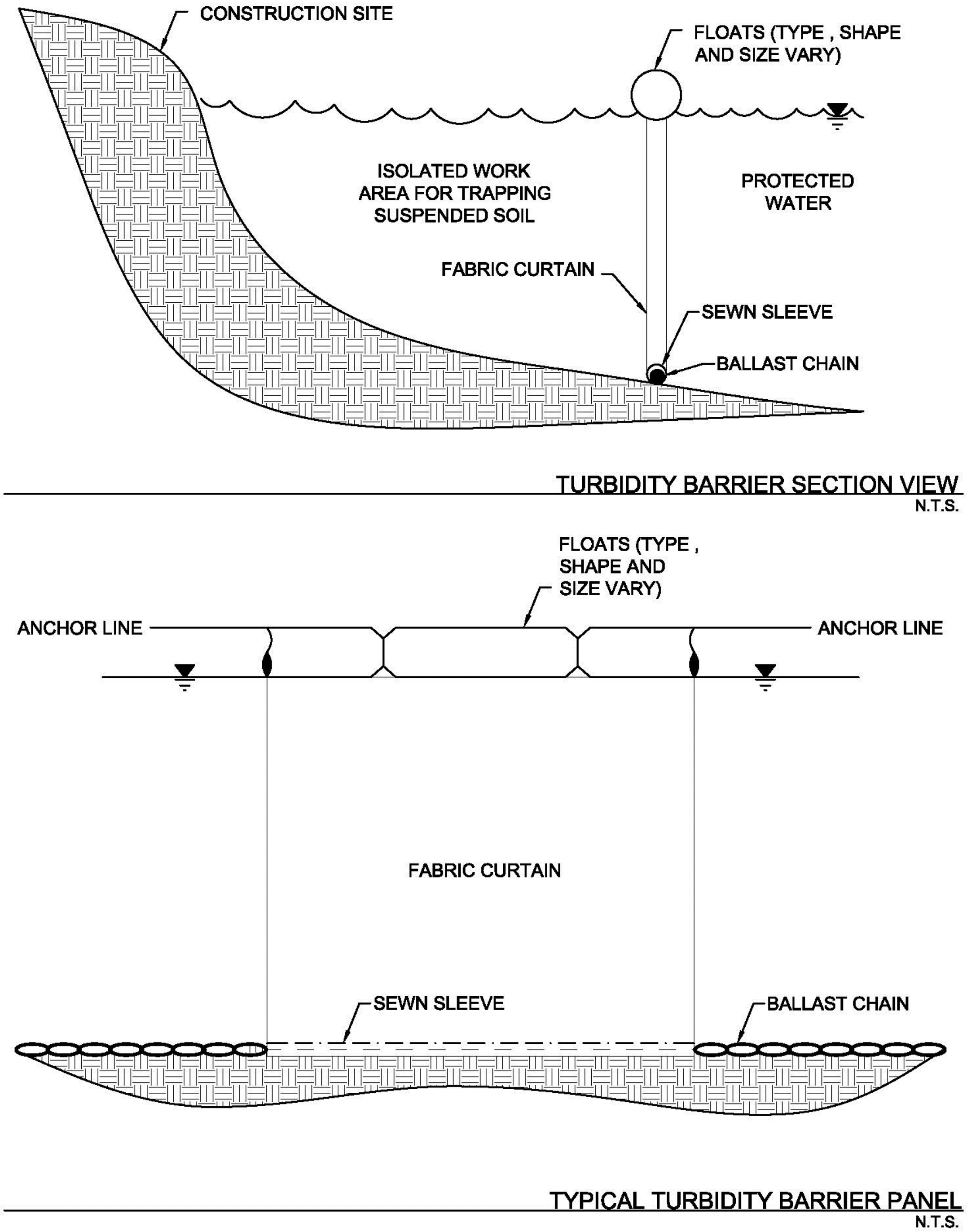
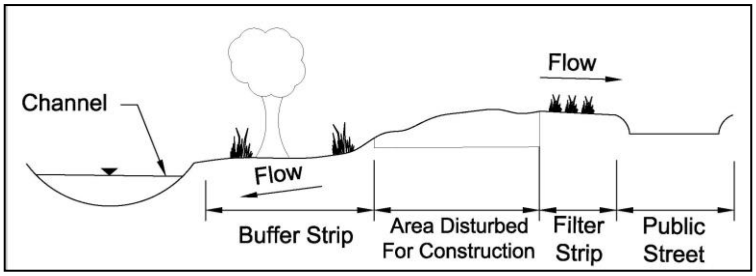


Figure 3.34 Schematics of Turbidity Barrier

3.15 Vegetated Filter Strips and Buffers

Sediment Control



Description: Buffer strips (existing vegetation) and filter strips (planted vegetation) are sections of vegetated land adjacent to disturbed areas. They are designed with low slopes to convey sheet flow runoff from disturbed areas, resulting in the removal of sediment and other pollutants as the runoff passes through vegetation and infiltration occurs.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum width (direction of flow across the vegetation) dependent on slope of disturbed area
- Maximum ratio of disturbed area to vegetated area dependent on slope
- Existing vegetation must meet criteria for type and coverage
- Dense grass required for planted vegetation
- Demarcate limits of vegetation and protect from traffic

ADVANTAGES / BENEFITS:

- Effective secondary control for removing clay particles
- Disperses flow and slows velocities to decrease erosion potential in receiving water
- Preserves the character of existing riparian corridor
- May become part of the permanent stormwater controls

DISADVANTAGES / LIMITATIONS:

- Appropriate as a primary control only for drainage areas of 2 acres or less and under certain site conditions
- Maximum 150 feet of flow to vegetated strip or buffer is used as a primary control
- Cannot treat large volumes or concentrated flows
- Not effective as a perimeter control when the perimeter cuts across contours instead of following contours
- Must limit access to vegetated portion of the site

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Rake accumulations of sediment from the vegetation
- Repair bare areas

TARGETED POLLUTANTS

- ● Sediment
- ◒ Nutrients & Toxic Materials
- ○ Oil & Grease
- ○ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=0.35-0.85

(*Depends on many conditions in addition to soil type*)

IMPLEMENTATION CONSIDERATIONS

- ◒ Capital Costs
- ○ Maintenance
- ○ Training
- ◒ Suitability for Slopes > 5%

Other Considerations:

- *Coordination with final landscaping*

3.15.1 Primary Use

Vegetated filter strips and buffers are used to reduce the velocity of sheet flow and reduce the volume of runoff through infiltration. In the process, sediment is removed as the runoff is filtered through the vegetation and infiltration occurs.

Vegetated filter strips and buffers are frequently used a secondary sediment control, since their performance is highly variable. They may be used as a primary sediment control only for small areas and under select site conditions.

3.15.2 Applications

Vegetated buffers are most applicable on development projects that are adjacent or near to floodplains, wetlands, streams and other natural waterways. Vegetated strips may be established along roads and property lines as a perimeter control for development. They are also applicable along the down slope side of utility line projects.

Vegetated buffers may be a primary sediment control for small areas where the conditions meet design criteria. They are also commonly used as a secondary control with other perimeter controls to provide higher levels of sediment removal. Vegetated areas have more capability to remove fine particle sizes than many conventional sediment controls. Combinations such as an organic filter tube or silt fence at the upslope edge of a vegetated strip are very effective.

In addition to perimeter control, vegetated strips are applicable for slope protection. Strips may be established at regular intervals to interrupt long or steep slopes. The strips maintain sheet flow, decrease velocities, and decrease erosion on the slopes.

3.15.3 Design Criteria

Vegetated buffers should be preserved along existing floodplains, wetlands, channels, and other natural waters whenever possible, even when the buffer is not a primary sediment control. Check for local requirements, as many municipalities mandate a vegetated buffer to maintain the character of the riparian corridor along a natural waterway. Vegetated buffers are encouraged to protect existing waterways by decreasing velocities, dispersing flow, and attenuating volume before the runoff reaches the waterway. If the development plans necessitate disturbing the riparian corridor, phase the development (when possible) to retain a vegetated buffer until final grading and landscaping at the end construction.

The evaluation and use of vegetated strips and buffers for use as a sediment control are unique to each site. The designer should carefully consider slope, vegetation, soils, depth to impermeable layer, depth to ground water, and runoff sediment characteristics before specifying a vegetated strip or buffer as a primary sediment control. This consideration is especially true for buffer strips of existing vegetation. If the buffer is not correctly planned, the first storm event can damage the natural vegetation beyond repair.

Design criteria in this section are only applicable when a vegetated strip or buffer is intended to be a primary or secondary sediment control for the construction site. As discussed above, a vegetated buffer may be preserved for other reasons that do not necessitate the use of these criteria if other sediment controls are provided for the construction site.

General

- Maximum slope of the vegetated strip or buffer shall be 5% across the width of the vegetation in the direction of flow.
- To maintain sheet flow, maximum distance of flow to the vegetated filter shall be 150 feet.
- Vegetated buffers and strips may only serve as a primary sediment control when the contributing drainage area has a slope of 15% or less. On steeper slopes, another perimeter control (e.g. organic filter tube, silt fence) may be installed at the upslope edge of the vegetated buffer or strip as a primary control, with the vegetation serving as a secondary control.

- Maximum disturbed area contributing runoff to the vegetated strip or buffer shall be 2 acres.
- Vegetated filter strips and buffers shall be a minimum of 15 feet wide. Width shall be increased based on the slope of the disturbed area as shown in the following table. Although the slope of the disturbed area may be up 15%, the slope of the vegetated strip or buffer is still limited to 5% maximum if used as a primary control for sediment.

Table 3.10 Sizing of Vegetated Buffers and Strips

Maximum Slope of Contributing Drainage Area	*Maximum Ratio of Disturbed Area to Vegetated Area*	*Minimum Width of Vegetated Area (Direction of Flow)*
5%	8:1	15 feet
10%	5:1	30 feet
15%	3:1	50 feet

- Access to vegetated buffers and strips shall be prohibited. These areas shall be protected from all traffic. No activities should occur in these areas, including no parking of the workers' vehicles, no eating of lunch, etc.
- Install controlled and stabilized ingress/egress points to manage traffic and direct it away from vegetation. Fence the vegetation or provide other means of protection to prevent vehicles and equipment from driving on the vegetated areas.
- Vegetated buffers and filter strips should not be used when high ground water, shallow depth to bedrock, or low soil permeability will inhibit infiltration of runoff.

Buffers of Existing Vegetation

- Fencing, flagged stakes spaced at a maximum of 6 feet, or other measures shall be used to clearly mark existing vegetation that is being preserved as a buffer before the start of any clearing, grubbing, or grading.
- Existing vegetation must be well established to be used as a vegetated buffer. It may be a mix of trees, sapling/shrubs, vines and herbaceous plants. However, the herbaceous plants shall cover at least 80 percent of the ground area.
- Bare soil shall not be visible within the buffer. Area between herbaceous plants shall be covered with a natural litter of organic matter (e.g. leaves, dead grass).
- Lots with a thick stand of existing grasses may preserve strips of the grasses as perimeter control in addition to using vegetation as a buffer along a natural waterway.

Strips of Planted Vegetation

- Vegetated strips should only be used when the site perimeter is along (parallel to) contours. Erosion of the vegetated strip will be a problem when the strip is placed along roads or site perimeters that cut across contours, resulting in runoff flowing along, instead of across, the filter strip.
- Minimize vehicle and equipment traffic and other activities that could compact soils on areas that will be planted for vegetated strips.
- Sod is required when the strip is intended to immediately function as a sediment control.
- Erosion control blankets (ECBs) should be used to prevent erosion and provide sediment control while establishing vegetation for a filter strip. If ECBs are not used, than another perimeter control is required until the vegetation is mature. Refer to *Section 2.3 Erosion Control Blankets*.
- Refer to the *Section 2.9 Vegetation* for criteria on establishing vegetation.
- When using vegetated strips for slope protection, spacing of the strips should be designed based on

slope steepness and type of soil. The strips may be planted directly on the slope grade when the slope is flatter than 2:1. For slopes of 2:1 and steeper, vegetation should be established on terraces. Terraces shall have a transverse slope of 1 percent in the opposite direction of the slope (i.e. back into the ground).

3.15.4 Design Guidance and Specifications

Guidance for analysis of the hydraulic loading on filter strips is in *Section 13.3 of the Stormwater Controls Technical Manual*.

No specification for vegetated filter strips and buffers is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.15.5 Inspection and Maintenance Requirements

Vegetated filter strips and buffers should be inspected regularly (at least as often as required by the TPDES Construction General Permit). If rill erosion is developing, additional controls are needed to spread the flow before it enters the vegetated area. Rake light accumulations of sediment from the vegetation. Remove trash that accumulates in the vegetation. Additional sediment controls (e.g. a line of organic filter tubes or silt fence), are needed if sediment accumulations are large enough to bury the vegetation.

Inspect established planted vegetation for bare areas and place sod or install seeded erosion control blankets, as appropriate. Mow as needed after planted vegetation is mature.

3.15.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

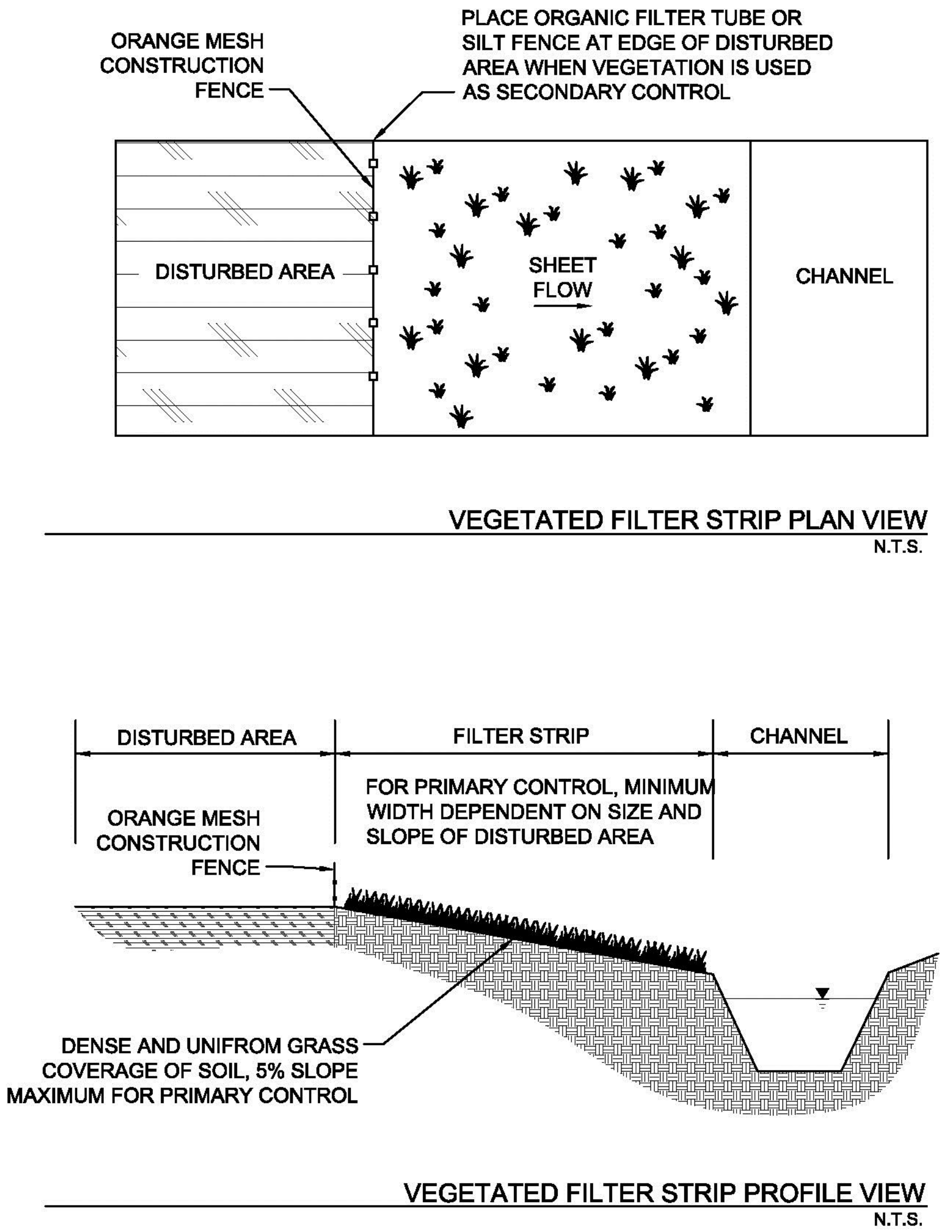
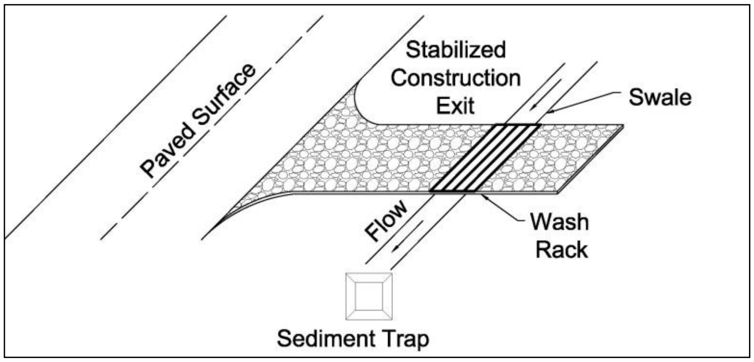


Figure 3.35 Schematics of Vegetated Filter Strip

3.16 Wheel Cleaning Systems

Sediment Control



Description: Wheel cleaning systems are used with a stabilized construction exit to remove soil from vehicle wheels and undercarriages prior to leaving the construction site. The cleaning system may be as simple as uneven, steel racks that "rumble" the vehicle or as complex as a pre-manufactured wash bay. Systems that include wash water must provide for collecting the water and removing sediments and other pollutants prior to discharge.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Locate within the stabilized construction exit
- Design according to type of soil and the number and size of vehicles using the cleaning system
- Provide a means of collecting wash water and removing sediment before discharge

ADVANTAGES / BENEFITS:

- Effectively reduces off-site sediment tracking
- Components of the system may be re-used on different projects

DISADVANTAGES / LIMITATIONS:

- Requires separate construction entrances and exits
- Requires frequent cleaning to remain functional
- Effectiveness dependent on operator training
- Sediment trapping controls won't remove oils or other pollutants in the wash water
- Potential overflows and discharges of wash water if sediment controls not carefully designed for the maximum amount of wash water to be generated

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Remove sediment from wheel cleaning device before sediment accumulates to half the depth of the device
- Remove sediment from sediment traps before it reaches a depth of half the design depth or 12 inches, whichever is less
- Dewater and clean wash basins using dewatering controls

TARGETED POLLUTANTS

- ● Sediment
- ○ Nutrients & Toxic Materials
- ○ Oil & Grease
- ○ Floatable Materials
- ○ Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

Fe=N/A

IMPLEMENTATION CONSIDERATIONS

- ● Capital Costs
- ● Maintenance
- ◒ Training
- ○ Suitability for Slopes > 5%

Other Considerations:

- *Management of wash water*
- *Prohibitions on the discharge of soaps and petroleum products*

3.16.1 Primary Use

Wheel cleaning systems are used to remove soil from construction vehicles and equipment before they leave the site and enter paved streets. Wheel cleaning systems are used with a stabilized construction exit to minimize the tracking of soil from disturbed areas. They provide added protection and reduce the need to remove sediment from streets.

3.16.2 Applications

Wheel cleaning systems can be used on any construction site where a stabilized construction exit is not adequate to prevent off-site tracking of soil. However, because of their cost, they are most applicable for:

- Sites with large areas (> 10 acres) that are disturbed for long periods of time;
- Sites with a large number of vehicles and/or heavy equipment entering and exiting the site, which that will quickly and repeatedly degrade rock exits;
- Sites with clay soils or wet site conditions that result in tires accumulating large amounts of soil; or
- Sites where contaminated soils might be present.

3.16.3 Design Criteria

General

- Provide separate entrances and exits to the construction site so that incoming vehicles do not drive through the wheel cleaning system. Signage and employee training is critical to making the system work.
- Wheel cleaning systems should be located within the stabilized construction exit so that the vehicle does not pick up additional sediment load by traversing disturbed areas. A minimum of 25 feet of stabilized exit shall be maintained between the cleaning system and the paved road.
- The stabilized exit shall be sloped at 1 percent toward the cleaning system.
- The width of the stabilized exit may be reduced to 10 to 20 feet, depending on the size and number of vehicles using the exit, as long as all exiting traffic is funneled through the cleaning system.
- Post a sign requiring all vehicles to use the cleaning system before leaving the site. Posted speed limit through the wheel cleaning system should be 5 mph.
- Wheel cleaning systems should be designed with ease of access to areas where sediment will accumulate, so the system can be frequently cleaned.

Rumble Racks

- The minimum cleaning system shall consist of 10 foot wide, 8 foot long, steel grates with individual bars of the grates at varying heights to shake the vehicle and knock off soil. These grates are also known as rumble racks.
- Minimum length of the rumble rack shall be the length of the circumference of the largest tire on vehicles that will be using the construction exit. Two to three lengths of grates are typically necessary to provide adequate soil removal, depending on soil type and size of vehicles.
- Grates shall be placed over an excavated pit that is a minimum of one foot deep.
- Grates may be purchased pre-made from vendors or constructed by welding 10 foot lengths of structural steel tubing (rectangular section) or angle. The lengths of steel ("bars" of the rumble rack) should be welded to steel beams or other cross supports in a manner that provides for alternating heights. This is accomplished with rectangular steel tube by alternating the long and short sides of

the tube upward. Angle iron, welded to the support structure with the angle pointed upward, may also be used. Round tubing shall not be used, as it does not adequately shake the tires.

- Size and spacing of bars and support beams shall be designed based on the size and weight of vehicles expected to be using the rumble rack.
- Welded or manufactured grates may be cleaned and re-used on multiple projects.

Wheel Washes

- Two common types of wheel wash systems constructed onsite are the corrugated metal wheel wash and the flooded basin wheel wash. In addition, several companies manufacture packaged wash systems that can be assembled onsite and re-used. All of these require a source of water, and several of the packaged systems require electricity to run pumps for water pressure.
- All wheel washes must provide a means to collect the wash water in a sediment basin or other sediment control that provides equivalent or better treatment prior to discharge from the site.
- For the flooded basin wheel wash, sedimentation occurs in the wash basin, meaning the basin cannot be used for a period of time while settling is allowed to occur. Cleaning of the basin should be done first thing in the morning after particles have settled overnight, and ideally the basin would be cleaned on Monday after settling all weekend. If the basin is pumped for cleaning, it should be accomplished using the controls in *Section 3.3 Dewatering Controls*.
- Corrugated metal wheel washes shall be constructed over a drainage swale that conveys the wash water to a sediment barrier, typically a sediment basin. However, a passive or active treatment system may be needed to adequately remove suspended solids depending on the permit requirements for the site.
- Swales, sediment basins, stone outlet sediment traps, and other controls for the wash water must be sized for the anticipated flows from the wheel wash using criteria in their respective sections of this manual. Depending on the volume of water, two sediment controls may be needed in parallel, to allow for settling and cleaning of one sediment control while the other is in operation for the wheel wash.
- Manufactured wash systems frequently collect, filter, and recycle the wash water, resulting in the use of less water and producing less wash water to treat for sediment removal. For this reason, they may be more cost-effective over the life of the project, even if their initial cost is higher.
- If a packaged wheel wash system does not include a sediment collection area, then a swale and sediment trap is required, similar to the corrugated metal wheel wash.
- Prohibit the use of soap for wheel washing. The purpose of a wheel wash is to remove soil that would otherwise fall off on the roadway, not to clean the vehicle. Refer to *Section 4.10 Vehicle and Equipment Management* for proper vehicle washing procedures. The discharge of wash water with soap in it is prohibited, and soap is not removed by a sediment control.
- Train employees to only use water in the wheel wash for removing accumulations of soil from the wheels and undercarriage. Minimize water contact with other portions of the vehicle or equipment. Wash water contaminated with oil, grease or fuel requires special handling and disposal. Refer to *Section 4.10 Vehicle and Equipment Management*.

3.16.4 Design Guidance and Specifications

No specification for construction of this item is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

3.16.5 Inspection and Maintenance Requirements

Wheel cleaning systems should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Systems should be cleaned frequently, at least weekly and sometimes daily, to ensure proper operation. Grated systems should be cleaned before sediment accumulates to half the depth of the pit below the grates. Depending on volume of traffic, flooded basin systems often needed daily pumping, cleaning and refilling to be effective.

The sediment basin or other sediment trapping device shall be inspected for damaged areas and repaired as necessary. Sediment that has accumulated in the wash water sediment control (must be removed before it reaches half the design depth of the device or 12 inches, whichever is less. The removed sediment shall be stockpiled or redistributed to areas of the site that are protected by erosion and sediment controls.

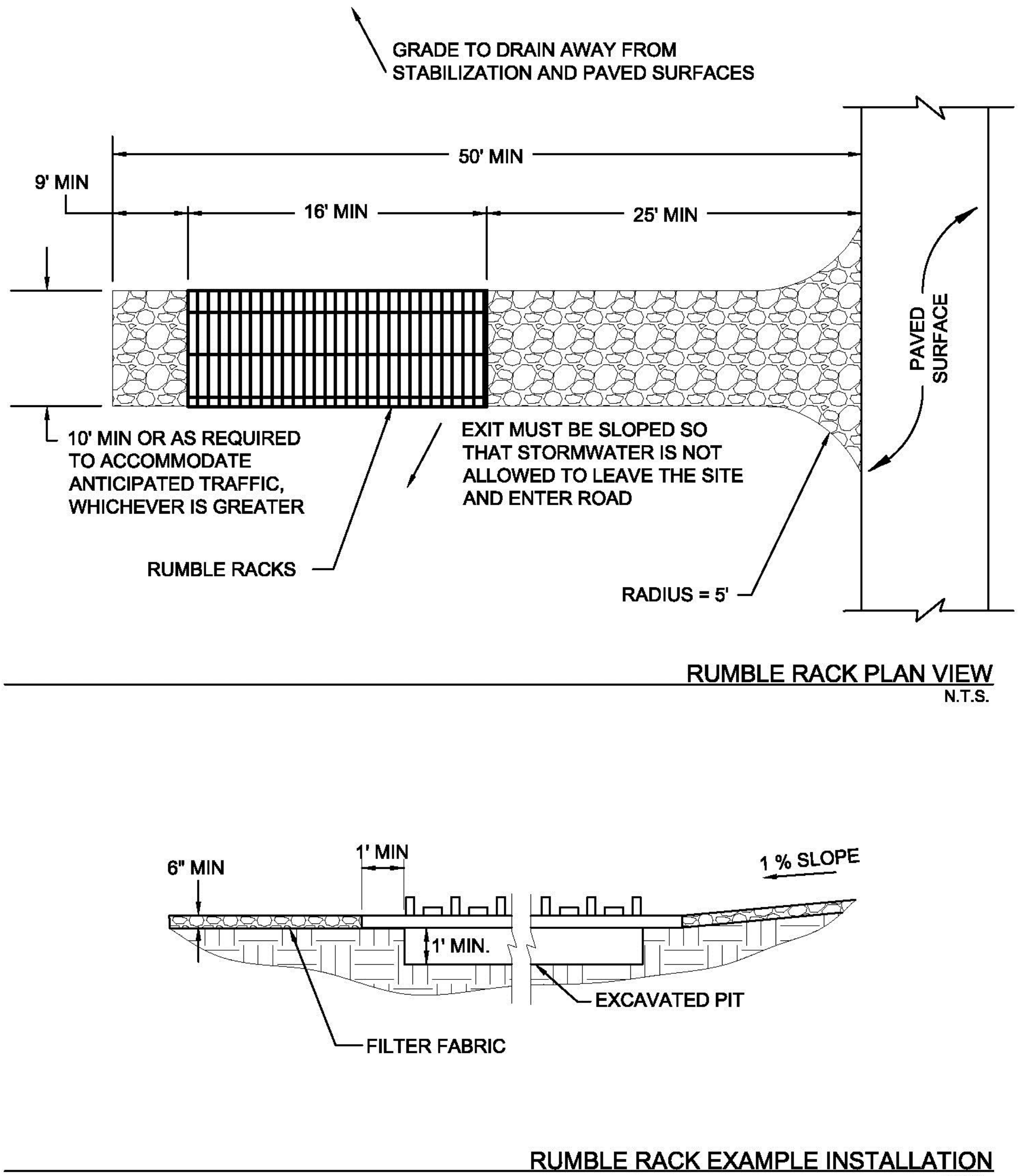
Water that ponds in the sediment basin should be inspected for sheen. If sheen is present, the water is considered contaminated by a petroleum product. Regulations of the TCEQ require this water to be pumped into containers and disposed of appropriately. It is not an authorized discharge from the construction site. Proper vehicle and equipment maintenance is essential to preventing this problem from occurring.

Manufacturer's recommendations should be followed for cleaning and maintaining packaged wheel wash systems.

3.16.6 Example Schematics

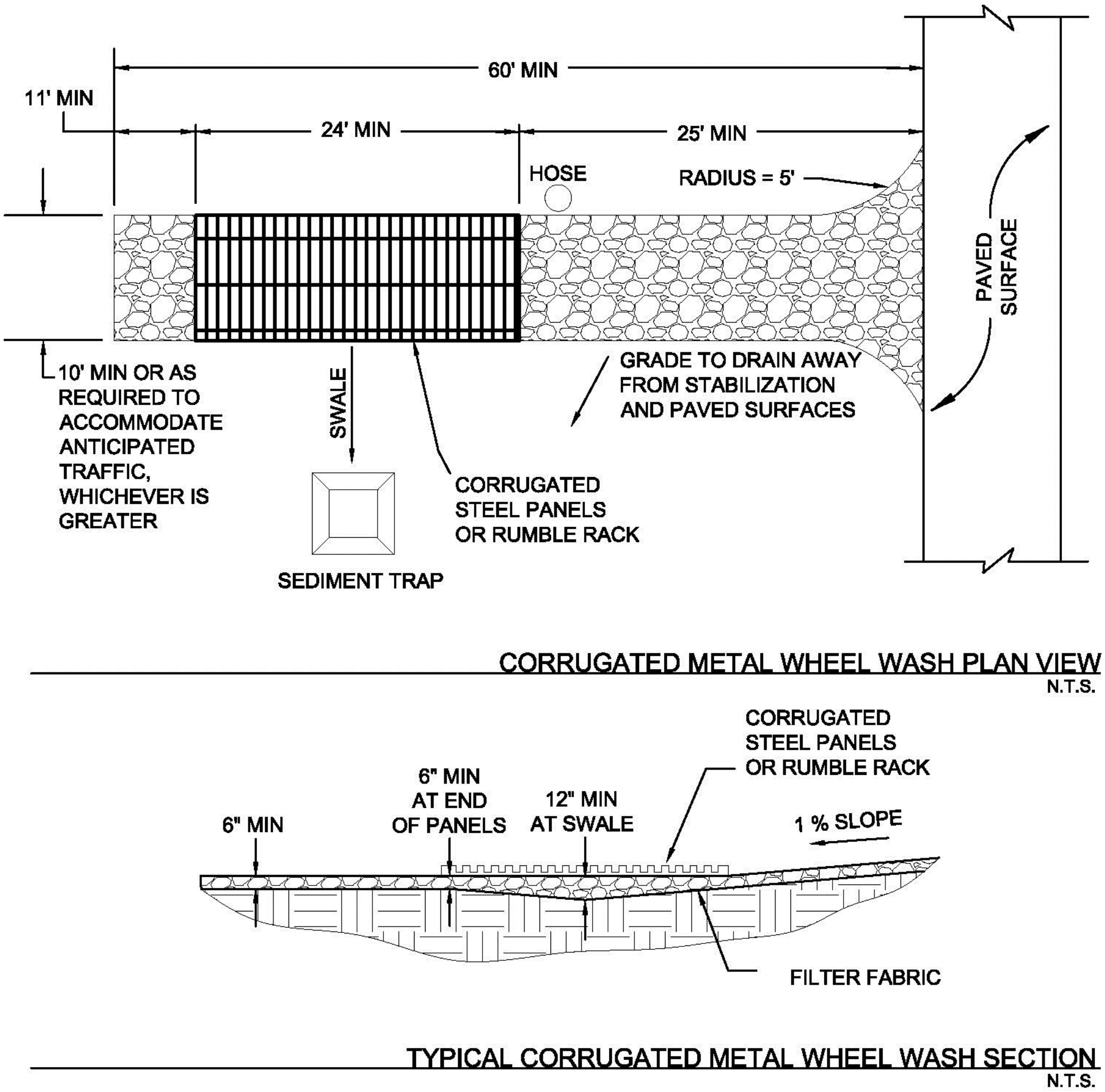
The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.



NOTE: DESIGNER SHALL SPECIFY SIZE AND SPACING OF THE STEEL BARS AND SUPPORT BEAMS BASED ON SIZE AND WEIGHT OF VEHICLES EXPECTED TO BE EXITING THE CONSTRUCTION SITE.

Figure 3.36 Schematics of Rumble Rack Wheel Cleaning



NOTE: PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED FOR THE DESIGN.

- NUMBER AND TYPE OF VEHICLES USING THE SYSTEM DAILY
- SOURCE OF WASH WATER
- VOLUME OF WASTE WASH WATER GENERATED DAILY
- SIZE AND TYPE OF SEDIMENT TRAP
- DESIGN PARAMETERS FOR THE SWALE
- PARAMETERS REQUIRED FOR DESIGN OF THE SEDIMENT TRAP
- ESTIMATE OF WHEEL WASH DOWN-TIME FOR CLEANING THE SEDIMENT TRAP

Figure 3.37 Schematics of Corrugated Metal Wheel Wash
(Source: Modified from California Stormwater Quality Association BMP Handbook BMP Detail TC-1)

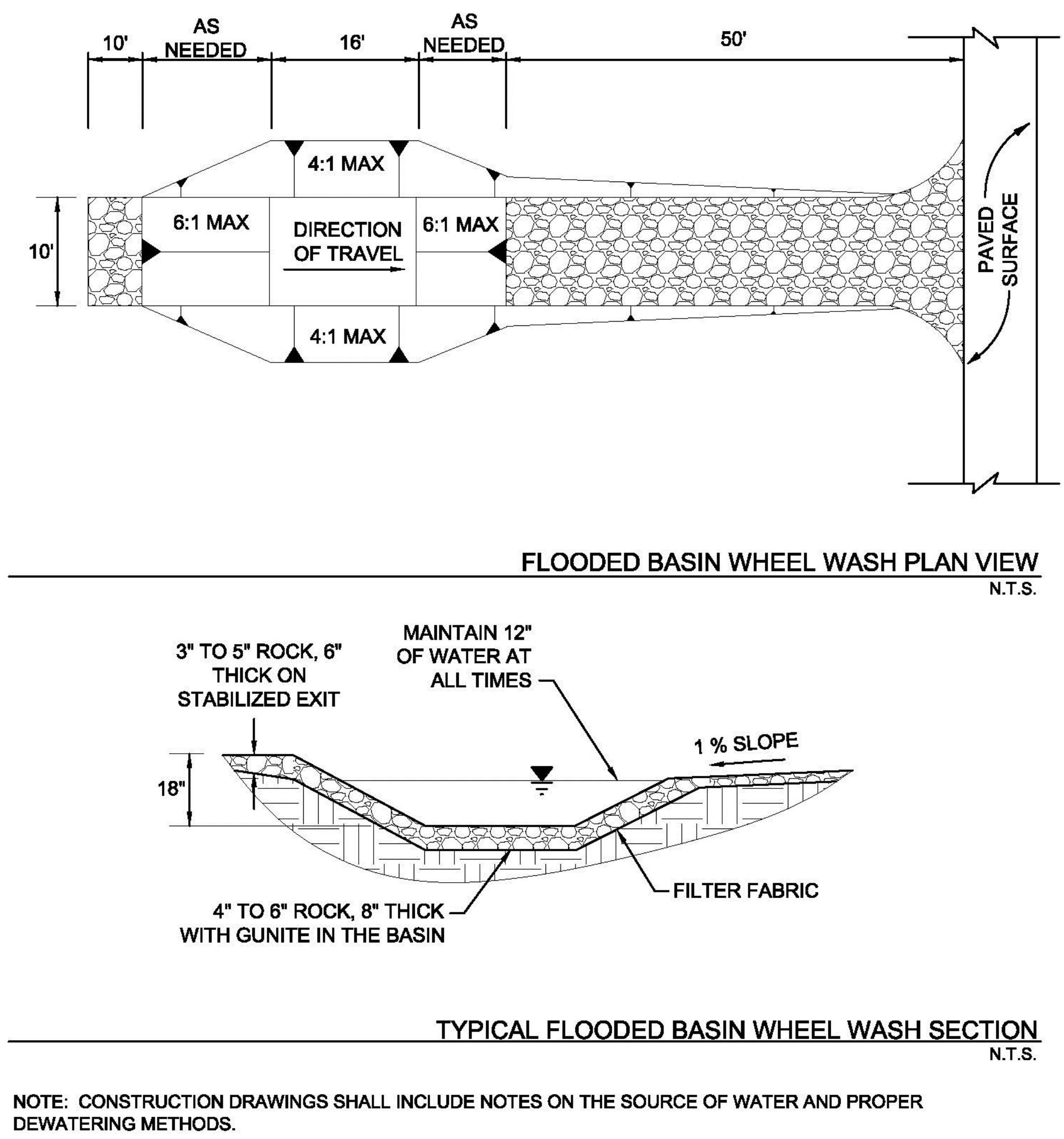


Figure 3.38 Schematics of Flooded Basin Wheel Wash
(Source: Modified from Oregon Department of Environmental Quality Erosion and Control Sediment Manual Detail SC-11)

4.0 Material and Waste Controls

4.1 Chemical Management

Material and Waste Control

Description: Chemical management addresses the potential for stormwater to be polluted with chemical materials and wastes that are used or stored on a construction site. The objective of chemical management is to minimize the potential of stormwater contamination by construction chemicals through appropriate recognition, handling, storage, and disposal practices.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designate a person responsible for chemical management
- Minimize the amount of chemicals and waste stored onsite
- Provide secondary containment that's 110 percent of the largest container in the containment
- Label all containers
- Prohibit the discharge of washout water
- Train workers in proper procedures
- Provide timely removal of waste materials

LIMITATIONS:

- Not intended to address site-assessment and pre-existing contamination
- Does not address demolition activities and potential pre-existing materials, such as lead and asbestos
- Does not address contaminated soils
- Does not address spill and leak response procedures
- Does not address chemicals associated with vehicle and equipment management

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Check for proper storage and evidence of leaks and spills
- Make sure all containers are labeled
- Check waste containers and dispose of the waste when 90 percent full
- Verify procedures are being followed
- Train new employees and regularly re-train all employees

TARGETED POLLUTANTS

○ Sediment
● Nutrients & Toxic Materials
● Oil & Grease
○ Floatable Materials
● Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management (boxed)
Housekeeping Practices (boxed)

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
● Training
○ Suitability for Slopes > 5%

Other Considerations:

- *TCEQ regulations for hazardous waste*

4.1.1 Primary Use

These management practices, along with applicable OSHA, EPA, and TCEQ requirements, are implemented at construction sites to prevent chemicals, hazardous materials, and their wastes from becoming stormwater pollutants.

4.1.2 Applications

Chemical management is applicable on all construction sites where chemicals and hazardous materials are stored or used and could result in pollutants being discharged with stormwater. Many chemicals, such as paints, grease, concrete curing compounds, and pesticide are present at most construction sites. Chemical management is most effective when used in conjunction with controls in *Section 4.8 Spill and Leak Response Procedures*.

Management of vehicle and equipment maintenance chemicals is applicable to all construction activities. These chemicals are the most common ones on construction sites; plus, there are specific stormwater permit requirements for vehicle and equipment maintenance. For these reasons, the management of chemicals associated with vehicles and equipment are found in *Section 4.10 Vehicle and Equipment Maintenance*.

Chemical management techniques are based on proper recognition, handling, and disposal practices by construction workers and supervisors. Key elements are education and modification of workers' behavior and provisions for safe storage and disposal. Cooperation and vigilance is required on the part of supervisors and workers to ensure that the procedures are followed.

The following list (not all inclusive) gives examples of targeted chemicals:

- Paints
- Solvents
- Stains
- Wood preservatives
- Cutting oils
- Greases
- Roofing tar
- Pesticides, herbicides, & fertilizers
- Concrete curing compound

It is not the intent of chemical management to supersede or replace normal site assessment and remediation procedures. Significant spills and/or contamination warrant immediate response by trained professionals. Chemical management shall be applied in combination with criteria in *Section 4.8 Spill and Leak Response Procedures*.

4.1.3 Design Criteria

- Construction plan notes shall require controls for all chemicals, hazardous materials, and their wastes that are potentially exposed to precipitation or stormwater runoff.
- Show the location of chemical and hazardous waste storage and secondary containment on the drawings, or require the contractor to add this information.
- The contractor should be required to designate a site superintendent, foreman, safety officer, or other senior person who is onsite daily to be responsible for implementing chemical management.
- Specify use of the least hazardous chemical to perform a task when alternatives are available. To the extent possible, do not use chemicals that are classified as hazardous materials or that will generate

a hazardous waste. A hazardous material is any compound, mixture, solution, or substance containing a chemical listed on the EPA's *Consolidated List of Chemicals Subject to the Emergency Planning and Community Right-to-Know Act (EPCRA) and Section 112(r) of the Clean Air Act* (EPA 550-B-01-003, October 2001), available at:

http://www.epa.gov/ceppo/pubs/title3.pdf

Chemical and Hazardous Material Storage

- As much as possible, minimize the exposure of building materials, building products, landscape materials, fertilizers, pesticides, herbicides, detergents, and other materials to precipitation and stormwater runoff.
- Chemicals and hazardous materials shall be stored in their original, manufacturers' containers, inside a shelter that prevents contact with rainfall and runoff.
- The amount of chemicals and hazardous materials stored onsite shall be minimized and limited to the materials necessary for the current phase of construction.
- Material Safety and Data Sheets (MSDSs) shall be available for all chemicals used or stored onsite.
- Chemical and hazardous materials shall be stored a minimum of 50 feet away from inlets, swales, drainage ways, channels, and other waters, if the site configuration provides sufficient space to do so. In no case shall material and waste sources be closer than 20 feet from inlets, swales, drainage ways, channels, and other waters.
- Use secondary containment controls for all hazardous materials. Containment shall be a minimum size of 110 percent of the largest chemical container stored within the containment.
- If an earthen pit or berm is used for secondary containment, it shall be lined with plastic or other material that is compatible with the chemical being stored.
- Chemical and hazardous material storage shall be in accordance with Federal and State of Texas regulations and with the municipality's fire codes.
- Storage locations shall have appropriate placards for emergency responders.
- Containers shall be kept closed except when materials are added or removed.
- Chemicals shall be dispensed using drip pans or within a lined, bermed area or using other spill/overflow protection measures.

Washout Procedures

- Many chemicals (e.g. stucco, paint, form release oils, curing compounds) used during construction may require washing of applicators or containers after use. The discharge of this wash water is prohibited.
- Wash water shall be collected in containers, labeled, and classified for correct waste disposal.
- A licensed waste hauler shall be used for wash water.

Chemical and Hazardous Waste Handling

- Ensure that adequate waste storage volume is available.
- Ensure that waste collection containers are conveniently located and compatible with the waste chemicals.
- Waste containers shall have lids and be emptied or hauled for disposal when they are 90 percent full or more frequently.
- Segregate potentially hazardous waste from non-hazardous construction waste and debris.

- Do not mix different chemical wastes. First, dangerous reactions may result. Second, all of the waste will be classified as the most hazardous waste in the container and will increase disposal costs.
- Clearly label all chemical and hazardous waste containers to identify which wastes are to be placed in each container.
- Based on information in the Material Safety Data Sheet, ensure that proper spill containment material is available onsite and maintained near the storage area.
- Do not allow potentially hazardous waste to be stored on the site for more than 90 days.
- Enforce hazardous waste handling and disposal procedures.

Disposal Procedures

- Regularly schedule waste removal to minimize onsite storage.
- Use only licensed waste haulers.
- For special and hazardous wastes, use licensed hazardous waste transporter that can classify, manifest and transport the special or hazardous wastes for disposal.
- Where possible, send wastes such as used oil to a recycler instead of a disposal facility.
- No chemical waste shall be buried, burned or otherwise disposed of onsite.

Education

- Instruct workers on safe chemical storage and disposal procedures.
- Instruct workers in identification of chemical pollutants and proper methods to contain them during storage and use.
- Educate workers of potential dangers to humans and the environment from chemical pollutants.
- Educate all workers on chemical storage and disposal procedures.
- Have regular meetings to discuss and reinforce identification, handling and disposal procedures (incorporate in regular safety seminars).
- Establish a program to train new employees.

Quality Control

- Designated personnel shall monitor onsite chemical storage, use, and disposal procedures.
- Educate and if necessary, discipline workers who violate procedures.
- Retain trip reports and manifests that document the recycling or disposal location for all chemical, special, and hazardous wastes that all hauled from the site.

4.1.4 Design Guidance and Specifications

National guidance for response procedures are established by the Environmental Protection Agency (EPA) in the Code of Federal Regulations (CFR). Specific sections addressing spills are governed by:

- 40 CFR Part 261 Identification and Listing of Hazardous Waste.
- 40 CFR Part 262 Standards Applicable to Generators of Hazardous Waste.
- 40 CFR Part 263 Standards Applicable to Transporters of Hazardous Waste.
- 49 CFR Parts 171-178 of the Transportation Hazardous Materials Regulations.

Guidance for storing, labeling, and managing hazardous waste in the State of Texas are established by the Texas Commission on Environmental Quality (TCEQ) in the Texas Administrative Code Title 30, Chapter 335, Industrial Solid Waste and Municipal Hazardous Waste.

No specification for chemical management measures is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.1.5 Inspection and Maintenance Requirements

Chemical management measures should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for proper storage and evidence of leaks or spills. Check that all chemicals, hazardous materials, and wastes are properly stored and labeled. If not stored properly, take corrective action, and reinforce procedures through re-education of employees.

If leaks or spills have occurred, check that proper clean up and reporting procedures have been followed. If procedures have not been followed, take corrective action. Check that all employees have been trained in spill and leak procedures as detailed in *Section 4.8 Spill and Leak Response Procedure*.

4.2 Concrete Sawcutting Waste Management

Waste Control

Description: Sawcutting of concrete pavement is a routine practice used to control shrinkage cracking immediately following placement of plastic concrete. It is also used to remove curb sections and pavement sections for pavement repairs, utility trenches, and driveways. Sawcutting for joints involves sawing a narrow, shallow grove in the concrete, while sawcutting for removals is usually done full depth through the slab. Water is used to control saw blade temperature and to flush the detritus from the sawed groove. The objective of concrete sawcutting waste management is to prevent the resulting slurry of process water and fine particles with its high pH from becoming a water pollutant.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Prohibit discharge of untreated slurry
- Educate employees on proper procedures
- Continuously vacuum slurry and cuttings during sawcutting operation
- Block inlets to prevent discharges
- Establish an onsite containment area (minimum 1 ft freeboard) if immediate disposal of the vacuumed slurry is not feasible
- Water evaporation and concrete recycling are the recommended disposal methods when slurry is not vacuumed

LIMITATIONS:

- Only one part of concrete waste management
- Does not address concrete demolition waste

MAINTENANCE REQUIREMENTS:

- Check for uncollected slurry after all sawcutting operations
- Inspect collection areas and repair containment as needed
- Dispose of sediment and cuttings when collection area volume is reduced by 50 percent
- Train new employees and regularly re-train all employees

TARGETED POLLUTANTS

○ Sediment
● Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
● Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

○ Capital Costs
◒ Maintenance
● Training
○ Suitability for Slopes > 5%

Other Considerations:

- *Coordinate with concrete waste management*

4.2.1 Primary Use

Pavement sawcutting is performed on almost all construction projects that include removal or installation of pavement. Properly managing the slurry and cuttings from sawcutting prevents them from affecting surface and ground water resources.

4.2.2 Applications

Concrete sawcutting waste management is applicable on construction activities where sawcutting is part of the work, regardless of the size of the total area disturbed. It is also applicable on repair and maintenance projects that may not be required to implement erosion and sediment controls.

Concrete sawcutting waste management is based on the proper collection and disposal of the slurry and cuttings. Employee education is critical to ensuring correct procedures are followed.

4.2.3 Design Criteria

- Construction plan notes shall include proper concrete sawcutting waste management procedures.
- The contractor should be required to designate the site superintendent, foreman, or other person who is responsible for concrete sawcutting to also be responsible for concrete sawcutting waste management.

Slurry Collection

- During sawcutting operations, the slurry and cuttings shall be continuously vacuumed or otherwise recovered and not be allowed to discharge from the site.
- If the pavement to be cut is near a storm drain inlet, the inlet shall be blocked by sandbags or equivalent temporary measures to prevent the slurry from entering the inlet. Remove the sandbags immediately after completing sawcutting operations, so they do not cause drainage problems during storm events.
- The slurry and cuttings shall not be allowed to remain on the pavement to dry out.

Slurry Disposal

- Develop pre-determined, safe slurry disposal areas.
- Collected slurry and cuttings should be immediately hauled from the site for disposal at a waste facility. If this is not possible, the slurry and cuttings shall be discharged into onsite containment.
- The onsite containment may be an excavated or bermed pit lined with plastic that is a minimum of 10 millimeters thick. Refer to *Section 4.3 Concrete Waste Management* for additional design criteria and an example schematic. If the project includes placement of new concrete, slurry from sawcutting may be disposed of in facilities designated for the washout of concrete trucks instead constructing a separate containment.
- The containment shall be located a minimum of 50 feet away from inlets, swales, drainage ways, channels, and other waters, if the site configuration provides sufficient space to do so. In no case shall the collection area be closer than 20 feet from inlets, swales, drainage ways, channels and other waters.
- Several, portable, pre-fabricated, concrete washout, collection basins are commercially available and are an acceptable alternative to an onsite containment pit.
- Remove waste concrete when the containment is half full. Always maintain a minimum of one foot freeboard.

- Onsite evaporation of slurry water and recycling of the concrete waste is the preferred disposal method. When this is not feasible, discharge from the collection area shall only be allowed if a passive treatment system is used to remove the fines. Criteria are in *Section 3.7 Passive Treatment System*. Mechanical mixing is required in the collection area. The pH must be tested, and discharge is allowed only if the pH does not exceed 8.0. The pH may be lowered by adding sulfuric acid to the slurry water. Dewatering of the collection area after treatment shall follow the criteria in *Section 3.3 Dewatering Controls*.
- Care shall be exercised when treating the slurry water for discharge. Monitoring must be implemented to verify that discharges from the collection area do not violate groundwater or surface water quality standards.
- Geotextile fabrics such as those used for silt fence should not be used to control sawcutting waste, since the grain size is significantly smaller than the apparent opening size of the fabric.
- Use waste and recycling haulers and facilities approved by the local municipality.

Education

- Supervisors must be made aware of the potential environmental consequences of improperly handling sawcutting slurry and waste.
- Train all workers performing sawcutting operations on the proper slurry and cuttings collection and disposal procedures.

4.2.4 Design Guidance and Specifications

No specification for concrete sawcutting waste management is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.2.5 Inspection and Maintenance Requirements

Concrete sawcutting waste management measures should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Project personnel should inspect the operations to assure that operators are diligent in controlling the water produced by the sawcutting activities. Pavement should be inspected each day after operations to ensure that waste removal has been adequately performed. Residual waste should be cleaned. Reinforce proper procedures with workers.

Inspect the collection area for signs of unauthorized discharges. Repair containment area as needed. Remove sediment and fines when the collection area volume is reduced by 50 percent.

4.3 Concrete Waste Management

Waste Control

Description: Concrete waste at construction sites comes in two forms: 1) excess fresh concrete mix, including residual mix washed from trucks and equipment, and 2) concrete dust and concrete debris resulting from demolition. Both forms have the potential to impact water quality through stormwater runoff contact with the waste. The objective of concrete waste management is to dispose of these wastes in a manner that protects surface and ground water.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Prohibit the discharge of untreated concrete washout water
- Prohibit dumping waste concrete anywhere except at pre-determined, regulated, recycling or disposal sites
- Provide a washout containment with a minimum of 6 cubic feet of containment volume for every 10 cubic yards of concrete placed
- Minimum 1 foot freeboard on containment
- Minimum 10 mil plastic lining of containment
- Washout water evaporation and concrete recycling are the recommended disposal methods
- Educate drivers and operators on proper disposal and equipment cleaning procedures

LIMITATIONS:

- Does not address concrete sawcutting waste

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Check for and repair any damage to washout containment areas
- Clean up any overflow of washout pits
- Regularly remove and properly dispose of concrete waste

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs

◒ Maintenance

● Training

○ Suitability for Slopes > 5%

Other Considerations:

- *None*

TARGETED POLLUTANTS

○ Sediment

● Nutrients & Toxic Materials

○ Oil & Grease

○ Floatable Materials

● Other Construction Wastes

4.3.1 Primary Use

Concrete waste management is used to prevent the discharge of concrete wash water and waste into stormwater runoff. A number of water quality parameters can be affected by the introduction of concrete, especially fresh concrete. Concrete affects the pH of runoff, causing significant chemical changes in water bodies and harming aquatic life. Suspended solids in the form of both cement and aggregated dust are also generated from both fresh and demolished concrete waste.

4.3.2 Applications

Concrete waste management is applicable to all construction sites where existing concrete is being demolished or new concrete is being placed, regardless of the size of the total area disturbed. It is also applicable on repair and maintenance projects that may not be required to implement erosion and sediment controls.

4.3.3 Design Criteria

- The discharge of washout water to an inlet, swale, or any portion of the storm drainage system or a natural drainage system (e.g. channel) shall be prohibited.
- Construction plan notes shall state that the discharge of concrete washout to anything except a designated containment area is prohibited.
- Show the location of the concrete washout containment on the drawings, or require the contractor to provide this information.
- The contractor should be required to designate the site superintendent, foreman, or other person who is responsible for concrete placement to also be responsible for concrete waste management.

Unacceptable Waste Concrete Disposal Practices

- Dumping in vacant areas on the job-site.
- Illicit dumping onto off-site lots or any other placed not permitted to receive construction demoliotion debris.
- Dumping into ditches, drainage facilities, or natural water ways.
- Using concrete waste as fill material or bank stabilization.

Recommended Disposal Procedures

- Identify pre-determined, regulated, facilities for disposal of solid concrete waste. Whenever possible, haul the concrete waste to a recycling facility. Disposal facilities must have a Class IV (or more stringent) municipal solid waste permit from the TCEQ.
- A concrete washout pit or other containment shall be installed a minimum of 50 feet away from inlets, swales, drainage ways, channels, and other waters, if the site configuration provides sufficient space to do so. In no case shall concrete washout occur closer than 20 feet from inlets, swales, drainage ways, channels and other waters.
- Provide a washout area with a minimum of 6 cubic feet of containment volume for every 10 cubic yards of concrete poured. Alternatively, the designer may provide calculations sizing the containment based on the number of concrete trucks and pumps to be washed out.
- The containment shall be lined with plastic (minimum 10 millimeters thick) or an equivalent measure to prevent seepage to groundwater.
- Mosquitoes do not typically breed in the high pH of concrete washout water. However, the concrete washout containment should be managed in a manner that prevents the collection of other water that could be a potential breeding habitat.

- Do not excavate the washout area until the day before the start of concrete placement to minimize the potential for collecting stormwater.
- Do not discharge any water or wastewater into the containment except for concrete washout to prevent dilution of the high pH environment that is hostile to mosquitoes.
- Remove the waste concrete and grade the containment closed within a week of completing concrete placement. Do not leave it open to collect stormwater.
- If water must be pumped from the containment, it shall be collected in a tank, neutralized to lower the pH, and then hauled to a treatment facility for disposal. Alternatively, it may be hauled to a batch plant that has an onsite collection facility for concrete washout water.
- Do **not** pump water directly from the containment to the Municipal Separate Storm Sewer System or a natural drainage way without treating for removal of fine particles and neutralization of the pH.

- Multiple concrete washout areas may be needed for larger projects to allow for drying time and proper disposal of the washout water and waste concrete.
- Portable, pre-fabricated, concrete washout containers are commercially available and are an acceptable alternative to excavating a washout area.
- Evaporation of the washout water and recycling of the concrete waste is the preferred disposal method. After the water has evaporated from the washout containment, the remaining cuttings and fine sediment shall be hauled from the site to a concrete recycling facility or a solid waste disposal facility.
- Remove waste concrete when the washout containment is half full. Always maintain a minimum of one foot freeboard.
- Use waste and recycling haulers and facilities approved by the local municipality.
- When evaporation of the washout water is not feasible, discharge from the collection area shall only be allowed if a passive treatment system is used to remove the fines. Criteria are in *Section 3.7 Passive Treatment System*. Mechanical mixing is required within the containment for passive treatment to be effective. The pH must be tested, and discharge is allowed only if the pH does not exceed 8.0. The pH may be lowered by adding sulfuric acid to the water. Dewatering of the collection area after treatment shall follow the criteria in *Section 3.3 Dewatering Controls*.
- Care shall be exercised when treating the concrete washout water for discharge. Monitoring must be implemented to verify that discharges do not violate groundwater or surface water quality standards.
- On large projects that are using a nearby batch plant, a washout facility associated with the plant and under the plant's TPDES Multi-Sector General Permit may be used instead of installing an onsite containment area for truck washout.

Education

- Drivers and equipment operators should be instructed on proper disposal and equipment washing practices (see above).
- Supervisors must be made aware of the potential environmental consequences of improperly handled concrete waste.

Enforcement

- The construction site manager or foreman must ensure that employees and pre-mix companies follow proper procedures for concrete disposal and equipment washing.
- Employees violating disposal or equipment cleaning directives must be re-educated or disciplined if necessary.

Demolition Practices

- Monitor weather and wind direction to ensure concrete dust is not entering drainage structures and surface waters.
- Spray water on structures being demolished to wet them before start of demolition operations. Reapply water whenever dust is observed.
- Construct sediment traps or other types of sediment detention devices downstream of demolition activities to capture and treat runoff from demolition wetting operations.

4.3.4 Design Guidance and Specifications

No specification for concrete waste management is currently available in the Standard Specifications for Public Works – North Central Texas Council of Governemtns.

4.3.5 Inspection and Maintenance Requirements

Concrete waste management controls should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for proper handling of concrete waste. Check concrete washout pits and make repairs as needed. Washout pits should not be allowed to overflow. Maintain a schedule to regularly remove concrete waste and prevent over-filling.

If illicit dumping of concrete is found, remove the waste and reinforce proper disposal methods through education of employees.

4.3.6 Example Schematics

The following schematics are example applications of the construction control. They are intended to assist in understanding the control's design and function.

The schematics are **not for construction**. They may serve as a starting point for creating a construction detail, but they must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

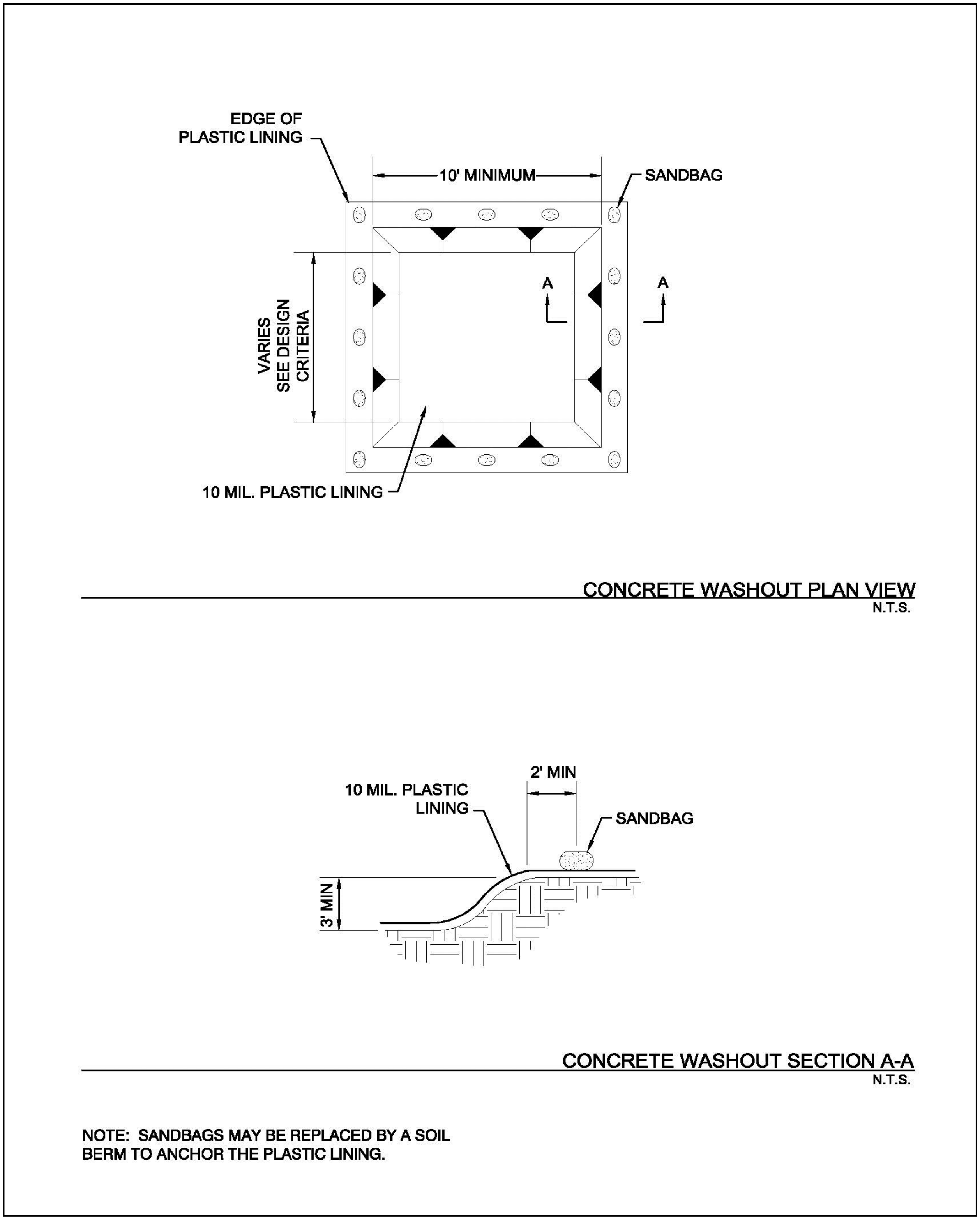


Figure 4.1 Schematics of Concrete Washout Containment

4.4 Debris and Trash Management

Waste Control

Description: Large volumes of debris and trash are often generated at construction sites, including packaging, pallets, wood waste, personal trash, scrap material, and a variety of other wastes. The objective of debris and trash management is to minimize the potential of stormwater contamination from solid waste through appropriate storage and disposal practices. Recycling of construction debris is encouraged to reduce the volume of material to be disposed of and associated costs of disposal.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Implement a job-site waste handling and disposal education and awareness program
- Provide sufficient and appropriate waste storage containers
- Provide timely removal of stored solid waste materials
- Train workers and monitor compliance

LIMITATIONS:

- Only addresses non-hazardous solid waste
- One part of a comprehensive construction site waste management program

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Empty waste containers regularly
- Clean up loose trash and debris daily
- Verify procedures are being followed
- Train new employees and regularly re-train all employees

TARGETED POLLUTANTS

○ Sediment
● Nutrients & Toxic Materials
○ Oil & Grease
● Floatable Materials
● Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
$\boxed{\textbf{Waste Management}}$
$\boxed{\textbf{Housekeeping Practices}}$

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
● Training
○ Suitability for Slopes > 5%

Other Considerations:

- *None*

4.4.1 Primary Use

Debris and trash management is used to minimize floatables and other wastes in stormwater. By controlling the trash and debris onsite, stormwater quality is improved and the need for extensive clean up upon completion of the project is reduced.

4.4.2 Applications

Debris and trash management is applicable on all construction sites where workers are present. Even if the only construction activity is earthwork, workers will still have drink bottles, lunch bags, and other wastes that must be managed.

Solid waste management for construction sites is based on proper storage and disposal practices by construction workers and supervisors. Key elements of the program are education and modification of improper disposal habits. Cooperation and vigilance is required on the part of supervisors and workers to ensure that the procedures are followed.

The following are lists describing the type of targeted materials.

- Construction (and Demolition) Debris:

    Dimensional lumber

    Miscellaneous wood (pallets, plywood, etc)

    Copper (pipe and electrical wiring)

    Miscellaneous metal (studs, pipe, conduit, sheathing, nails, etc)

    Insulation

    Brick and mortar

    Shingles

    Roofing materials

    Gypsum board

- Trash:

    Paper and cardboard (packaging, containers, wrappers)

    Plastic (packaging, bottles, containers)

    Styrofoam (cups, packing, and forms)

    Food and beverage containers

    Food waste

4.4.3 Design Criteria

- Construction plan notes shall include proper debris and trash management procedures.
- Show the location of waste storage containers on the drawings, or require the contractor to add this information.
- The contractor should be required to designate a site superintendent, foreman, safety officer, or other senior person who is onsite daily to be responsible for implementing debris and trash management.

Storage Procedures

- All waste sources and storage areas shall be located a minimum of 50 feet away from inlets, swales, drainage ways, channels and other waters, if the site configuration provides sufficient space to do so.

In no case shall material and waste sources be closer than 20 feet from inlets, swales, drainage ways, channels, and other waters.

- Construction waste and trash shall be stored in a manner that minimizes its exposure to precipitation and stormwater runoff.
- Whenever possible, minimize production of debris and trash.
- Instruct construction workers in proper debris and trash storage and handling procedures.
- Segregate potentially hazardous waste from non-hazardous construction site debris. Hazardous waste shall be managed according to the criteria in *Section 4.1 Chemical Management*.
- Segregate recyclable or re-usable construction debris from other waste materials. A goal of re-using or recycling 50 percent of the construction debris and waste is recommended.
- Keep debris and trash under cover in either a closed dumpster or other enclosed trash container that limits contact with rain and runoff and prevents light materials from blowing out.
- Check the municipality's storage requirements. Some municipalities have specific requirements for the size and type of waste containers for construction sites.
- Do not allow trash containers to overflow. Do not allow waste materials to accumulate on the ground.
- Prohibit littering by workers and visitors.
- Police site daily for litter and debris.
- Enforce solid waste handling and storage procedures.

Disposal Procedures

- If feasible, recycle construction and demolition debris such as wood, metal, and concrete.
- Trash and debris shall be removed from the site at regular intervals that are scheduled to empty containers when they are 90 percent full or more frequently.
- General construction debris may be hauled to a licensed construction debris landfill (typically less expensive than a sanitary landfill).
- Use waste and recycling haulers/facilities approved by the local municipality.
- No waste, trash, or debris shall be buried, burned or otherwise disposed of onsite.
- Cleared trees and brush may be burned if authorized by the municipality and proper permits are obtained from the county and/or TCEQ. Chipping of trees and brush for use as mulch is the preferred alternative to burning or offsite disposal.

Education

- Educate all workers on solid waste storage and disposal procedures.
- Instruct workers in identification of solid waste and hazardous waste.
- Have regular meetings to discuss and reinforce disposal procedures (incorporate in regular safety seminars).
- Clearly mark on all debris and trash containers which materials are acceptable.

Quality Control

- Foreman and/or construction supervisor shall monitor onsite solid waste storage and disposal procedures.
- Check the site, particularly areas frequented by workers during lunch and breaks, for loose trash and debris and the end of each work day.

- Discipline workers who repeatedly violate procedures.

4.4.4 Design Guidance and Specifications

No specification for debris and trash management measures is found currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.4.5 Inspection and Maintenance Requirements

Debris and trash management measures should be inspected regularly (at least as often as required by the TPDES Construction General Permit). If waste containers are overflowing, call the waste hauler immediately for a pick-up. If loose trash and debris are found around the site, reinforce proper waste management procedures through education of workers.

Construction sites must maintain separate waste containers clearly marked for non-hazardous, hazardous and recyclable waste. Check solid waste containers for chemical, special, or hazardous wastes that are improperly placed in them. These wastes shall be removed and handled according to criteria in *Section 4.1 Chemical Management*.

The site should be checked for loose litter and debris at the end of each working day.

4.5 Hyper-Chlorinated Water Management

Waste Control

Description: Hyper-chlorinated water is routinely used to disinfect new waterlines and appurtenances. Chlorine protects humans from pathogens in water, but it is toxic to aquatic ecosystems. The objective of hyper-chlorinated water management is to discharge the water in a manner that protects surface water and related aquatic ecosystems.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Educate employees on proper procedures
- Discharge to sanitary sewer if the system operator approves
- Discharge water onsite for natural chlorine attenuation
- Use appropriate dosage for chemical de-chlorination based on chemical used and chlorine concentration
- Chlorine concentration must be less than 4 ppm before leaving the site
- Use velocity dissipation devices for discharges
- Always monitor receiving waters for negative effects

LIMITATIONS:

- Discharge to sanitary sewer limited by sewer capacity
- Discharges limited to areas without vegetation that is to be preserved
- Wet, cool, and overcast days limits chlorine attenuation and removal

MAINTENANCE REQUIREMENTS:

- Monitor continuously during discharge
- Check for and repair any erosion caused by discharge
- Sample and test receiving water hourly for chlorine

TARGETED POLLUTANTS

○ Sediment
● Nutrients & Toxic Materials
○ Oil & Grease
○ Floatable Materials
○ Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
○ Maintenance
● Training
○ Suitability for Slopes > 5%

Other Considerations:

- *None*

4.5.1 Primary Use

Hyper-chlorinated water is used to disinfect new water lines.

4.5.2 Applications

Construction sites that install new water lines or repair or replace existing water lines should use hyper-chlorinated water management measures.

4.5.3 Design Criteria

- Drawing notes shall include procedures for the proper discharge of hyper-chlorinated water from waterline disinfection.
- The contractor should be required to designate the site superintendent, foreman, or other person who is responsible for water line disinfection to also be responsible for hyper-chlorinated water management.
- Educate employees about the environmental hazards of high chlorine concentrations and the proper procedures for handling hyper-chlorinated water.
- Hyper-chlorinated water shall not be discharged to the environment unless the chlorine concentration is reduced to 4 ppm or less by chemically treating to dechlorinate or by onsite retention until natural attenuation occurs.
- Water with a measurable chlorine concentration of less than 4 ppm is considered potable and an authorized discharge; however, large volumes of water with chlorine at this concentration can still be toxic to aquatic ecosystems. Do not discharge water that has been de-chlorinated to 4 ppm directly to surface water. It shall be discharged onto vegetation or through a conveyance system for further attenuation of the chlorine before it reaches surface water.
- Discharges of high flow rate and velocities shall be directed to velocity dissipation devices.

Discharge to Sanitary Sewers

- The preferred method of disposal for hyper-chlorinated water is discharge into a sanitary sewer system.
- Permission from the sanitary sewer operator **must** be obtained to discharge to the sanitary sewer.
- Limitations on discharges to the sanitary sewer are the capacity of the sanitary sewer and the availability of a sewer manhole near the construction site.
- The designer shall verify that the sanitary sewer is capable of receiving the flow rate that will result from dewatering the disinfected line within the required time.
- Consideration should be given to timing the discharge with the daily low flow period for the sanitary sewer system.

Onsite Discharge

- Hyper-chlorinated water may be applied to the construction site if it can be done without causing a discharge. The feasibility of this option is dependent on the volume of water, the size of the construction site, and the conditions of the site. Site application should not be done when the soil moisture content is high due to recent storm events.
- Chlorine can burn vegetation, so it should not be used to water vegetation that is being used for stabilization, vegetated filters or buffers, or other vegetation to be preserved.
- Hyper-chlorinated water may be discharged to an onsite retention area until natural attenuation occurs. The area may be a dry stormwater retention basin, or a portion of the site may be graded to form a temporary pit or bermed area.

- Natural attenuation of the chlorine may be aided by aeration. Air can be added to the water by directing the discharge over a rough surface (e.g. riprap) before it enters the temporary retention area or an aeration device (e.g. circulation pump) can be placed in the retention area.
- Onsite discharge may require several hours to a few days before the water is safe to discharge. The rate at which chlorine will attenuate is affected by soil conditions and weather conditions. Attenuation will occur quickest during warm, sunny, dry periods.
- If the hyper-chlorinated water is retained in a pit or basin, and then pumped to discharge, pumping shall follow the criteria in *Section 3.3 Dewatering Controls*.

Chemical Dechlorination

- If non-chemical means of dechlorination are not feasible, chemical methods may be used to neutralize the chlorine before discharging the hyper-chlorinated water.
- Vitamin C in the form of ascorbic acid or sodium ascorbate is the preferred dechlorination agent.
- Consider the National Fire Protection Association (NFPA) rating when selecting a dechlorination chemical. The NFPA rating is given by a series of three numbers ranging from 0 to 4, with 0 being no risk and 4 the highest risk. The sequence of numbers rank the health hazard, flammability risk and reactivity risk of the chemical. A NFPA rating of 0,0,0 indicates no risk for all three categories.
- Ensure appropriate personal protective equipment (PPE) is specified for workers depending on the chemical being used to neutralize the chlorine.
- The chemicals listed in Table 4.1 may be used to neutralize chlorine.

Table 4.1 Chemical Dechlorination Agents and Approximate Dosages

Dechlorinating Agent	*Dosing Rate (parts Agent : parts Chlorine)*	*Advantages*	*Disadvantages*
Ascorbic Acid (form of Vitamin C)	2.5:1	• Not toxic to aquatic species • Quick reaction time • NFPA rating of 0,0,0	• May lower pH in receiving water
Sodium Ascorbate (form of Vitamin C)	2.8:1	• Does not affect pH • Not toxic to aquatic species • Quick reaction time • NFPA rating of 0,0,0	• Greater amount needed than Ascorbic Acid • More expensive
Sodium Thiosulfate	2:1 to 7:1 depending on pH	• Less expensive • Readily available • Long history of use (familiarity)	• Must calculate dosage based on pH • Skin, eye, nose and throat irritant • Consumes oxygen in water • May encourage bacterial growth in receiving streams
Calcium Thiosulfate	1:1 to 0.5:1 depending on pH	• Less expensive • Not toxic to aquatic species • NFPA rating of 0,0,0	• Must calculate dosage based on pH • Over-dosing produces suspended solids • Over-dosing may increase turbidity in receiving water • May encourage bacterial growth in receiving streams

- The designer shall confirm dosages with the chemical supplier before using the dechlorination agent.

- Chlorine and residual agent concentrations and the pH of the discharged water shall be monitored at least hourly using field tests.
- The treated water should be discharged onto pavement or into a dry conveyance system to allow aeration and reaction time before the dechlorinated water reaches the receiving water. The receiving water should be closely monitored for any signs of negative effects from the discharge.

4.5.4 Design Guidance and Specifications

No specification for hyper-chlorinated water management is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.5.5 Inspection and Maintenance Requirements

Hyper-chlorinated water management measures should be monitored continuously while the hyper-chlorinated water is being discharged. Discharges to a sanitary sewer should be monitored for back-ups or overflows that indicate the discharge is exceeding the sewer's capacity. If these occur, the rate of discharge must be decreased or another discharge method is needed.

Onsite or chemically treated discharge should be monitored for chlorine and residual chemical concentrations. Verify that discharges are not causing erosion, and modify the discharge to use velocity dissipation devices if erosion is occurring. Repair any eroded areas. If water is being pumped from a temporary retention area, verify that appropriate dewatering controls are in place.

For all discharges, frequently inspect the receiving water for any evidence of negative effects. Sample and test the receiving water hourly for chlorine. Stop the discharge immediately if chlorine is detected and modify the discharge procedures before resuming.

4.6 Sandblasting Waste Management

Waste Control

Description: The objective of sandblasting waste management is to minimize the potential of stormwater quality degradation from sandblasting activities at construction sites. The key issues in this program are prudent handling and storage of sandblast media, dust suppression, and proper collection and disposal of spent media. It is not the intent of this control to outline all of the worker safety issues pertinent to this practice. Safety issues should be addressed by construction safety programs as well as local, state, and federal regulations.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Prohibit discharge of sandblasting waste
- Provide site specific fugitive dust control and containment equipment
- Educate employees on proper procedures
- Provide proper sandblast equipment for the job
- Ensure compliance by supervisors and workers

LIMITATIONS:

- Does not address hazardous materials that may be present in the waste
- Does not address spill and leak response procedures

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Contain and dispose of sandblast grit
- Train new employees and regularly re-train all employees

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

$\boxed{\textbf{Waste Management}}$

Housekeeping Practices

TARGETED POLLUTANTS

○ Sediment

● Nutrients & Toxic Materials

○ Oil & Grease

○ Floatable Materials

● Other Construction Wastes

IMPLEMENTATION CONSIDERATIONS

● Capital Costs

● Maintenance

● Training

○ Suitability for Slopes > 5%

Other Considerations:

- *OSHA requirements*
- *Special procedures for sandblasting operations on structures know to contain hazardous materials*
- *Possible site assessment or remediation required if hazardous materials present*

4.6.1 Primary Use

Sandblasting is typically used to clean a surface or prepare a surface for coatings. Since the sandblasting media consists of fine abrasive granules, it can be easily transported by running water. Sandblasting activities typically create a significant dust problem that must be contained and collected to prevent off-site migration of fines. Particular attention must be paid to sandblasting work on bridges, box culverts, and head walls that span or are immediately adjacent to streams and waterways.

4.6.2 Applications

This control should be implemented when sandblasting operations will occur on a construction site.

If a discharge of sandblasting waste occurs, it shall be considered a spill and handled according to the criteria in *Section 4.8 Spill and Leak Response Procedures*.

4.6.3 Design Criteria

- Construction plan notes shall include proper sandblasting waste management procedures.
- The contractor should be required to designate the site superintendent, foreman, or other person who is responsible for sandblasting to also be responsible for sandblasting waste management.
- Prohibit the discharge of sandblasting waste.

Operational Procedures

- Use only inert, non-degradable sandblast media.
- Use appropriate equipment for the job; do not over-blast.
- Wherever possible, blast in a downward direction.
- Install a windsock or other wind direction instrument.
- Cease blasting activities in high winds or if wind direction could transport grit to drainage facilities.
- Install dust shielding around sandblasting areas.
- Collect and dispose of all spent sandblast grit, use dust containment fabrics and dust collection hoppers and barrels.
- Non-hazardous sandblast grit may be disposed in permitted construction debris landfills or permitted sanitary landfills.
- If sandblast media cannot be fully contained, construct sediment traps downstream from blasting area where appropriate.
- Use sand fencing where appropriate in areas where blast media cannot be fully contained.
- If necessary, install misting equipment to remove sandblast grit from the air prevent runoff from misting operations from entering drainage systems.
- Use vacuum grit collection systems where possible.
- Keep records of sandblasting materials, procedures, and weather conditions on a daily basis.
- Take all reasonable precautions to ensure that sandblasting grit is contained and kept away from drainage structures.

Educational Issues

- Educate all onsite employees of potential dangers to humans and the environment from sandblast grit.

- Instruct all onsite employees of the potential hazardous nature of sandblast grit and the possible symptoms of over-exposure to sandblast grit.
- Instruct operators of sandblasting equipment on safety procedures and personal protection equipment.
- Instruct operators on proper procedures regarding storage, handling and containment of sandblast grit.
- Instruct operators and supervisors on current local, state and federal regulations regarding fugitive dust and hazardous waste from sandblast grit.
- Have weekly meetings with operators to discuss and reinforce proper operational procedures.
- Establish a continuing education program to indoctrinate new employees.

Materials Handling Recommendations

- Sandblast media should always be stored under cover away from drainage structures.
- Ensure that stored media or grit is not subject to transport by wind.
- Ensure that all sandblasting equipment and storage containers comply with current local, state and federal regulations.
- Refer to *Section 4.1 Chemical Management* if sandblast grit is known or suspected to contain hazardous components.
- Capture and treat runoff, which comes into contact with sandblasting material or waste.

Quality Assurance

- Foreman and/or construction supervisor should monitor all sandblasting activities and safety procedures.
- Educate and if necessary, discipline workers who violate procedures.
- Take all reasonable precautions to ensure that sandblast grit is not transported off-site or into drainage facilities.

4.6.4 Design Guidance and Specifications

No specification for sandblasting waste management is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.6.5 Inspection and Maintenance Requirements

Sandblasting waste management measures should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Verify that sandblasting grit is contained and disposed of properly. Check for downstream locations and the off-site perimeter for evidence of discharges or off-site transport by wind.

Check that daily records of sandblasting activities are current. Hold weekly meetings with operators to reinforce proper procedures. Regularly re-educate employees on potential dangers and hazards, safety procedures and proper handling.

4.7 Sanitary Waste Management

Waste Control

Description: The objective of sanitary waste management is to provide for collection and disposal of sanitary waste in a manner that minimizes the exposure to precipitation and stormwater. This is most often accomplished by providing portable facilities for construction site workers.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Provide sanitary facilities at the rate of one toilet per 10 workers for a 40-50 hour work week
- Locate portable toilets a minimum of 50 feet away from storm drain inlets, conveyance channels or surface waters
- If unable to meet the 50 foot requirement, locate portable toilets at least 20 feet away and provide secondary containment
- Show location of portable toilets on the drawings
- Have a plan to clean up spills

LIMITATIONS:

- Multiple facilities and/or facilities in several locations may be needed to adequately serve a construction site
- Facilities are subject to vandalism if not within a secured construction site

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Check for proper servicing, leaks and spills
- Service toilets at the frequency recommended by the supplier

TARGETED POLLUTANTS

- ○ Sediment
- ● Nutrients & Toxic Materials
- ○ Oil & Grease
- ○ Floatable Materials
- ● Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

- ○ Capital Costs
- ◒ Maintenance
- ◒ Training
- ○ Suitability for Slopes > 5%

Other Considerations:

- *None*

4.7.1 Primary Use

Sanitary facilities are used to properly store and dispose of sanitary wastes that are generated onsite.

4.7.2 Applications

Sanitary facilities should be available to workers at all construction sites. If permanent facilities are not available, portable toilets are placed at the construction site.

4.7.3 Design Criteria

- Construction plan notes shall include requirements for the contractor to provide an appropriate number of portable toilets based on the number of employees using the toilets and the hours they will work. The typical standard is one portable toilet per 10 workers for a 40-50 hour work week.
- The location of portable toilets shall be shown on the drawings.
- Sanitary facilities shall be placed a minimum of 50 feet away from storm drain inlets, conveyance channels or surface waters. If unable to meet the 50 foot requirement due to site configuration, portable toilets shall be a minimum of 20 feet away from storm drain inlets, conveyance channels or surface waters and secondary containment shall be provided in case of spills.
- The location of the portable toilets shall be accessible to maintenance trucks without damaging erosion and sediment controls or causing erosion or tracking problems.
- Sanitary facilities shall be fully enclosed and designed in a manner that minimizes the exposure of sanitary waste to precipitation and stormwater runoff.
- When high winds are expected, portable toilets shall be anchored or otherwise secured to prevent them from being blown over.
- The company that supplies and maintains the portable toilets shall be notified immediately if a toilet is tipped over or damaged in a way that results in a discharge. Discharged solid matter shall be vacuumed into the septic truck by the company that maintains the toilets. A solution of 10 parts water to 1 parts bleach shall be applied to all ground surfaces contaminated by liquids from the toilet.
- The operator of the municipal separate storm sewer system (MS4) shall be notified if a discharge from the portable toilets enters the MS4 or a natural channel.

4.7.4 Design Guidance and Specifications

No specification for sanitary facilities is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.7.5 Inspection and Maintenance Requirements

Sanitary facilities should be inspected regularly (at least as often as required by the TPDES Construction General Permit) for proper servicing, leaks and spills. Portable toilets shall be regularly serviced at the frequency recommended by the supplier for the number of people using the facility.

4.8 Spill and Leak Response Procedures

Waste Control

Description: Spill and leak response procedures address the management of spills and leaks that may occur at the construction site. The objective of the spill and leak response procedures is to minimize the discharge of pollutants from unplanned releases of chemicals, fuel, motor vehicle fluids, hazardous materials or wastes through appropriate recognition and response procedures.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Develop procedures based on the Material Safety and Data Sheets for substances onsite
- Maintain spill kits for petroleum products and other chemicals frequently onsite
- Post emergency contact numbers
- Designate a spill response coordinator
- Train employees
- Review reporting requirements for onsite chemicals

LIMITATIONS:

- Procedures susceptible to being forgotten because they are seldom or never used
- Larger spills and spills of extremely hazardous materials require special equipment and should be handled by professionals
- Not applicable to long-term contamination remediation

MAINTENANCE REQUIREMENTS:

- Review procedures regularly
- Verify spill kits, MSDSs, and emergency contacts are readily available
- Train new employees and regularly re-train all employees

TARGETED POLLUTANTS

○ Sediment
● Nutrients & Toxic Materials
● Oil & Grease
○ Floatable Materials
● Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
$\boxed{\textbf{Waste Management}}$
Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
● Training
○ Suitability for Slopes > 5%

Other Considerations:

- *OSHA, EPA and TCEQ regulations*

4.8.1 Primary Use

Spill and leak procedures are used to minimize the impact of accidental releases on surface water. Pollutants that are of concern for spill and leaks include chemicals, hazardous materials, fuel, motor vehicle fluids, washout waters, and wastes. Spill and leak response is a secondary control. Proper procedures for managing these pollutants should be the primary control and are the best way to prevent the need for spill and leak response.

4.8.2 Applications

Spill and leak response procedures are applicable on all construction sites where chemicals, hazardous materials, fuels, etc. are stored or used. They are most important when the construction site is adjacent or near to a floodplain, wetland, stream, or other waters.

4.8.3 Design Criteria

General

- An effective spill and leak response depends on proper recognition and response practices by construction workers and supervisors. Key elements are education and training.
- Records of releases that exceed the Reportable Quantity (RQ) for oil and hazardous substances should be maintained in accordance with the Federal and State regulations.
- Emergency contact information and spill response procedures shall be posted in a readily available area for access by all employees and subcontractors.
- Spill containment kits should be maintained for petroleum products and other chemicals that are regularly onsite. Materials in kits should be based on containment guidelines in the Material Safety and Data Sheets (MSDSs) for the substance most frequently onsite.
- Spill kits are intended for response to small spills, typically less than 5 gallons, of substances that are not extremely hazardous.
- Significant spills or other releases warrant immediate response by trained professionals.
- Suspected job-site contamination should be immediately reported to regulatory authorities and protective actions taken.

Coordinator

- The contractor should be required to designate a site superintendent, foreman, safety officer, or other senior person who is onsite daily to be the Spill and Leak Response Coordinator.
- The coordinator must have knowledge of and be trained in correct spill and leak response procedures.
- The coordinator shall be responsible for implementing the spill and leak procedures and training all employees and sub-contractors on the site-specific spill and leak procedures. The training should include their responsibility to immediately notify the coordinator if a spill or leak occurs.

Spill Response

- Upon discovery of a spill, employees and subcontractors shall implement the following procedures:
  - Immediately stop work and clear the area by moving upwind of the spill.
  - Remove all ignition sources.
  - Notify the Spill and Leak Response Coordinator.
  - If there is an immediate danger to health or life, contact 911.

- The Spill and Leak Response Coordinator shall perform the following when the spill is not immediately dangerous to health and safety:
  - Consult the MSDS for safety and response procedures.
  - If it can be done safely, use onsite spill kits and soil to contain the spill.
  - Notify a hazardous response company to remove and properly dispose of the spilled material and the contaminated containment materials.

Spill Reporting

- The Spill and Leak Response Coordinator is responsible for notifying authorities of spills and leaks. Notification requirements are based on Reportable Quantities as established by the type or material, quantity and location (onto land or into water in the state) of the release.
- Reportable Quantities (RQ) in the State of Texas are established by the TCEQ in Texas Administrative Code Title 30, Chapter 327 (30 TAC 327) Spill Prevention and Control.
- The Texas RQ for petroleum products and used oil is 25 gallons released onto land or any amount that causes sheen on water.
- Reportable Quantities for all other substances are listed in 30 TAC 327.4, which references the EPA List of Lists (EPA 550-B-01-003) available at: http://www.epa.gov/ceppo/pubs/title3.pdf
- The Spill and Leak Response Coordinator shall notify the following:
  - The municipality that operates the local Municipal Separate Storm Sewer System (MS4) if a spill or leak enters public rights-of-way or any type of drainage way or drainage infrastructure within the jurisdiction of the municipality.
  - State of Texas Spill Report Hotline at 1-800-832-8224 if the spill or leak exceeds the RQ; and during regular business hours, the TCEQ Dallas/Fort Worth Regional Office at 817-588-5800.
  - National Spill Response Center at 1-800-424-8802 if the spill or leak exceeds the RQ.

4.8.4 Design Guidance and Specifications

National guidance for response procedures are established by the Environmental Protection Agency (EPA) in the Code of Federal Regulations (CFR). Specific sections addressing spills are governed by:

- 40 CFR Part 68 Chemical Accident Prevention Provisions.
- 40 CFR Part 302 Designation, Reportable Quantities (RQ) and Notification.
- 40 CFR Part 355 Emergency Planning and Notification.

Guidance for emergency response procedures in the State of Texas are established by the Texas Commission on Environmental Quality (TCEQ) in the Texas Administrative Code Title 30, Chapter 327, Spill Prevention and Control.

No specification for construction of this item is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.8.5 Inspection and Maintenance Requirements

Spill and leak response measures should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Verify that spill containment materials are available for small spills. Also verify that emergency contact information is posted. These phone numbers and Material Safety and Data Sheets should be in a location that is readily accessible to workers.

If procedures are lacking, reinforce requirements by re-training employees.

4.9 Subgrade Stabilization Management

Material Control

Description: Lime and other chemicals are used extensively in the North Central Texas region to stabilize pavement subgrades for roadways, parking lots, and other paved surfaces, and as a subgrade amendment for building pad sites. These chemicals are applied to the soil and mixed through disking and other techniques, and then allowed to cure. The objective of subgrade stabilization management is to reduce the potential for runoff to carry the chemicals offsite, where they may impact aquatic life in streams, ponds, and other water bodies.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Educate employees on proper procedures
- Include procedural controls in stabilization specifications
- Limit stabilization operations to that which can be thoroughly mixed and compacted by the end of each workday
- Prohibit vehicle traffic, other than water trucks and mixing equipment, from passing over the area being stabilized until mixing is completed
- Avoid applications when there is a significant probability of rain that will produce runoff
- Roughen areas adjacent and downstream of stabilized areas to intercept lime from runoff
- Provide secondary containment according to *Section 4.1 Chemical Management* for stabilizers stored onsite

LIMITATIONS:

- Prevention of contamination is only effective method
- Does not address spill response when discharge occurs

MAINTENANCE REQUIREMENTS:

- Inspect down slope perimeters and outfalls regularly during stabilization operations
- Immediately halt operations if a discharge is found and modify procedures to prevent future discharges

TARGETED POLLUTANTS

○ Sediment

◒ Nutrients & Toxic Materials

○ Oil & Grease

○ Floatable Materials

○ Other Construction Wastes

APPLICATIONS

Perimeter Control

Slope Protection

Sediment Barrier

Channel Protection

Temporary Stabilization

Final Stabilization

Waste Management

Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

○ Capital Costs

◒ Maintenance

◒ Training

○ Suitability for Slopes > 5%

Other Considerations:

- *Chemical management controls for onsite storage of stabilization chemicals*

4.9.1 Primary Use

This measure should be implemented when chemicals are required for soil stabilization. Lime is the most commonly used for stabilization and is considered a chemical. Other agents may also be used for subgrade stabilization depending on the soil and site conditions.

4.9.2 Applications

Chemical stabilization can be used under a variety of conditions. The engineer should determine the applicability of chemical stabilization based on site conditions such as available open space, quantity of area to be stabilized, proximity of nearby water courses and other measures employed at the site. The use of diversion dikes and interceptor swales (see appropriate sections) to divert runoff away from areas to be stabilized can be used in conjunction with these techniques to reduce the potential impact of discharges from chemical stabilization.

Management of stabilization chemicals is based on implementing procedures to prevent a discharge. If a discharge occurs, it shall be considered a spill and handled according to the criteria in *Section 4.8 Spill and Leak Response Procedures*.

4.9.3 Design Criteria

- Construction plan notes or stabilization shall include procedural controls to minimize the discharge of chemical stabilizers.
- The contractor shall limit the amount of stabilizing agent onsite to that which can be thoroughly mixed and compacted by the end of each workday.
- Stabilizers shall be applied at rates that result in no runoff.
- Stabilization shall not occur immediately before and during rainfall events.
- No traffic other than water trucks and mixing equipment shall be allowed to pass over the area being stabilized until after completion of mixing the chemical.
- Areas adjacent and downstream of stabilized areas shall be roughened to intercept chemical runoff and reduce runoff velocity.
- Geotextile fabrics such as those used for silt fence should not be used to treat chemical runoff, because the chemicals are dissolved in the water and won't be affected by a barrier and the suspended solids are significantly smaller than the apparent opening size of the fabric.
- For areas in which phasing of chemical staibilization is impractical, a curing seal (such as Liquid Asphalt, Grace MC-250, or MC-800) applied at a rate of 0.15 gallons per square yard of surface can be used to protect the base.
- Use of sediment basins with a significant (>36 hour) drawdown time is encouraged to capture any accidental lime or chemical overflows when large areas are being stabilized (*Section 3.9 Sediment Basin*).
- Provide containment around chemical storage, loading and dispensing areas.
- If soil stabilizers are stored onsite, they shall be considered hazardous material and shall be managed according to the criteria in *Section 4.1 Chemical Management* to capture any accidental lime or chemical overflow.

4.9.4 Design Guidance and Specifications

No specification for subgrade stabilization management is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.9.5 Inspection and Maintenance Requirements

Subgrade stabilization operation should be observed frequently as the operations proceed for evidence of discharges. Inspect the down slope perimeter and all outfalls for evidence of discharges. Pay particularly attention to the outfall of drainage pipes connected to inlets within the area being stabilized. If a discharge is found, immediately halt stabilization operations until additional controls can be implemented.

4.9.6 Example Schematic

The following schematic is an example application of the construction control. It is intended to assist in understanding the control's design and function.

The schematic is **not for construction**. It may serve as a starting point for creating a construction detail, but it must be site adapted by the designer. In addition, dimensions and notes appropriate for the application must be added by the designer.

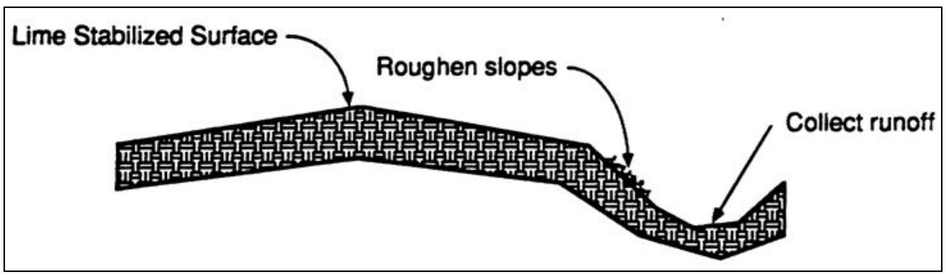


Figure 4.2 Schematic of Controls for Subgrade Stabilization

4.10 Vehicle and Equipment Management

Material and Waste Control

Description: Vehicle and equipment management addresses the practices associated with proper use and maintenance of vehicles and equipment at construction sites. The objective is to minimize the discharge of pollutants from vehicle and equipment operation, fueling, maintenance, and washing.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Prohibit the discharge of maintenance fluids and wash water with soap
- If feasible, prohibit onsite vehicle washing
- If feasible, prohibit onsite maintenance except fueling
- Provide secondary containment that's 110 percent of the largest container in the containment
- Use spill/overflow devices for fueling
- Never leave a fueling operation unattended
- Label all waste containers
- Train workers in proper procedures

LIMITATIONS:

- Cost of maintenance, repairs, and spill prevention equipment
- One part of a comprehensive construction site waste management program
- Does not address spill and leak response procedures

MAINTENANCE REQUIREMENTS:

- Inspect regularly
- Check for signs of leaks and spills and take corrective actions
- Place drip pans under leaking vehicles and equipment when parked
- Verify procedures are being followed
- Train new employees and regularly re-train all employees

TARGETED POLLUTANTS

○ Sediment
● Nutrients & Toxic Materials
● Oil & Grease
○ Floatable Materials
● Other Construction Wastes

APPLICATIONS

Perimeter Control
Slope Protection
Sediment Barrier
Channel Protection
Temporary Stabilization
Final Stabilization
Waste Management
Housekeeping Practices

IMPLEMENTATION CONSIDERATIONS

◒ Capital Costs
◒ Maintenance
● Training
○ Suitability for Slopes > 5%

Other Considerations:

- *None*

4.10.1 Primary Use

Vehicle and equipment management is used to minimize the pollutants that enter stormwater from fueling and maintenance activities.

4.10.2 Applications

Vehicle and equipment management is applicable on every construction site. The management controls are most effective when used in conjunction with controls in *Section 4.8 Spill and Leak Response Procedures*.

The management techniques are based on proper recognition and handling of pollutant sources related to vehicles and equipment. Key elements are education, established procedures, and provisions for safe storage and disposal of wastes. The following list (not all inclusive) gives examples of the targeted materials:

- Fuels
- Lube Oils
- Antifreeze
- Solvents
- Wash water

4.10.3 Design Criteria

- Construction plan notes shall state that the discharge of fuels, oils, or other pollutants used in vehicle and equipment operation and maintenance is prohibited.
- Construction plan notes shall state that the discharge of soaps or solvents used in vehicle and equipment washing is prohibited.
- On the construction plans, show the location of fuel tanks, motor vehicle fluids storage, and waste storage, including secondary containment, or require the contractor to provide this information.
- Provide secondary containment for fuel, new and waste oil, and other maintenance fluids that are stored onsite. Secondary containment shall have a minimum volume of 110 percent of the largest container within the containment.
- Criteria for the response to spills of motor vehicle fluids are in *Section 4.8 Spill and Leak Response Procedures*.
- The contractor should be required to designate a site superintendent, foreman, safety officer, or other senior person, who is on the site daily, to be responsible for implementing vehicle and equipment management.

Vehicle Washing

- Minimize the potential for the discharge of pollutants from equipment and vehicle washing by prohibiting these activities onsite, if practical. Vehicles and equipment should be transported to a commercial vehicle wash facility with appropriate discharge controls.
- Designate a wash area if vehicle and equipment washing must be done onsite. Require all washing to be done at this location. The area shall be graded so that all wash water flows to a sediment basin or other sediment control that provides equivalent or better treatment.
- Do not use soap for vehicle and equipment washing. Sediment controls will not remove soap from the wash water.

- Vehicle and equipment wash water may contain oils, greases, and heavy metals. Treatment to remove these pollutants is needed in addition to sediment trapping. Any wash water that has sheen on it must be considered polluted and cannot be discharged from the site without appropriate treatment. State or local discharge permits may be required.

Maintenance

- If possible, prohibit onsite maintenance except for fueling. Otherwise, limit onsite maintenance to routine preventive maintenance.
- Maintenance fluids should be stored in appropriate containers (closed drums or similar) and under cover.
- The ground under vehicles and equipment parked onsite should be inspected for drips and leaks before each use. Drip pans should be placed under parked vehicles and equipment that leak or drip.
- Vehicles and equipment that leak or drip should be removed from the site for repair as soon as possible.
- Vehicles and equipment that become inoperative should be removed from the site for repairs.

Fueling

- Check the municipality's requirements for fuel tanks. Some municipalities have specific requirements for the type of tank and secondary containment. At a minimum, local fire codes apply.
- Fuel should be dispensed using a drip pan or other spill/overflow device or within containment berms or other secondary containment.
- If the containment control is an earthen pit or berm, the containment shall be lined with plastic.
- If an automatic pump is used for fueling, it should be equipped with an overfill protection device.
- Workers performing fueling operations shall be trained in the correct procedures for fueling and spill response.
- Workers performing fueling operations shall be present and observe the fueling at all times. Fueling shall not be left unattended.
- A spill containment kit shall be maintained within 25 feet of the fueling area.

Waste Handling and Disposal

- Ensure that adequate waste storage volume is available.
- All waste containers shall be clearly labeled.
- Handling and disposal of waste from vehicle and equipment maintenance should be according to the criteria in *Section 4.1 Chemical Management*.

Education

- Instruct workers on procedures for washing, maintaining, and fueling vehicles and equipment.
- Instruct workers in identification of pollutants associated with vehicles and equipment.
- Have regular meetings to discuss and reinforce procedures (incorporate into regular safety briefings).
- Establish a continuing education program to train new employees.

4.10.4 Design Guidance and Specifications

No specification for vehicle and equipment management is currently available in the Standard Specifications for Public Works Construction – North Central Texas Council of Governments.

4.10.5 Inspection and Maintenance Requirements

Vehicle and equipment management controls should be inspected regularly (at least as often as required by the TPDES Construction General Permit). Verify that washing, fueling, storage, and disposal procedures are being followed. Correct workers where needed.

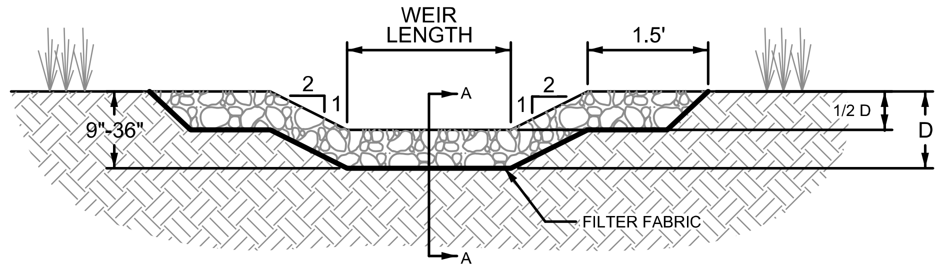
Fueling and maintenance fluid storage areas should be checked for signs of leakage or spills. If evidence is found, corrective actions should be implemented. Reinforce proper procedures through re-education of employees. Inspect areas where vehicles and equipment are parked for signs of leaks. Use drip pans where needed.

iSWM Construction Control Standard Details

Addendum to: **iSWM Technical Manual – Construction Controls**
The following are a selection of 10 iSWM construction control BMP schematics chosen to be provided in standard details.

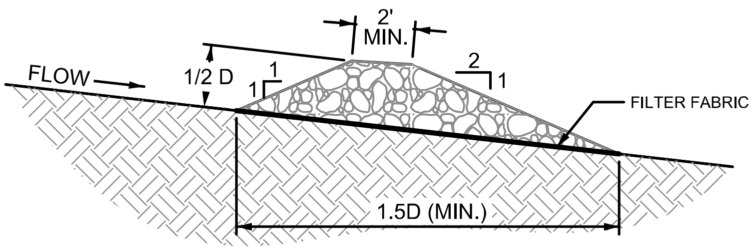
1. Rock Check Dams
2. Temporary Erosion Control Blankets
3. Dewatering Controls
4. Filter Tube Curb Inlet Protection
5. Hog Wire Weir Curb Inlet Protection
6. Curb Rock Sock On-Grade Curb Inlet Protection
7. Filter Tube Area Inlet Protection
8. Sediment Basin with Overflow Riser
9. Silt Fence
10. Stabilized Construction Exit

Created: January 2018



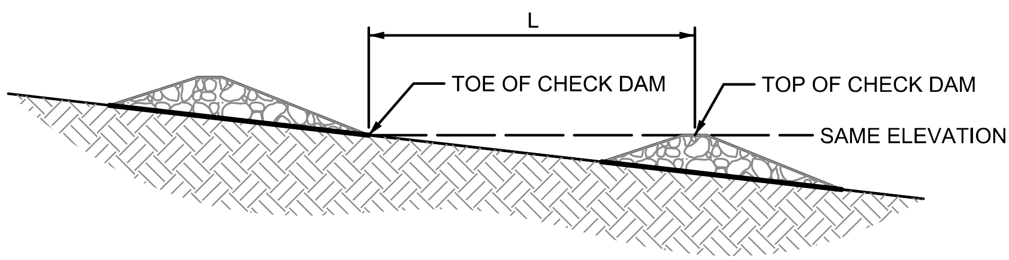
ROCK CHECK DAM VIEW LOOKING UPSTREAM

N.T.S.



CROSS SECTION A-A

N.T.S.



SPACE BETWEEN ROCK CHECK DAMS

N.T.S.

NOTES:
ACTUAL DIMENSIONS OF THE CHECK DAMS SHALL BE DESIGNED BASED ON FLOW CONDITIONS IN THE DRAINAGE SWALE OR DITCH. PROVIDE CALCULATIONS THAT DOCUMENT THE FOLLOWING PARAMETERS USED TO DESIGN THE CHECK DAM.

-HEIGHT OF CHECK DAMS (D) BASED ON SWALE OR DITCH DIMENSIONS AND FLOW CONDITIONS

-SPACING OF CHECK DAMS BASED ON GRADE OF THE SWALE OR DITCH. TOP OF DOWNSTREAM DAM SHALL BE AT THE SAME ELEVATION AS TOE OF UPSTREAM DAM

FIGURE 2.1 STANDARD CONSTRUCTION DETAIL - ROCK CHECK DAMS (1 OF 2)

ROCK CHECK DAM GENERAL NOTES:

1. SEE NCTCOG STANDARD SPECIFICATIONS (2017), SECTION 202.9 CHECK DAM (ROCK).

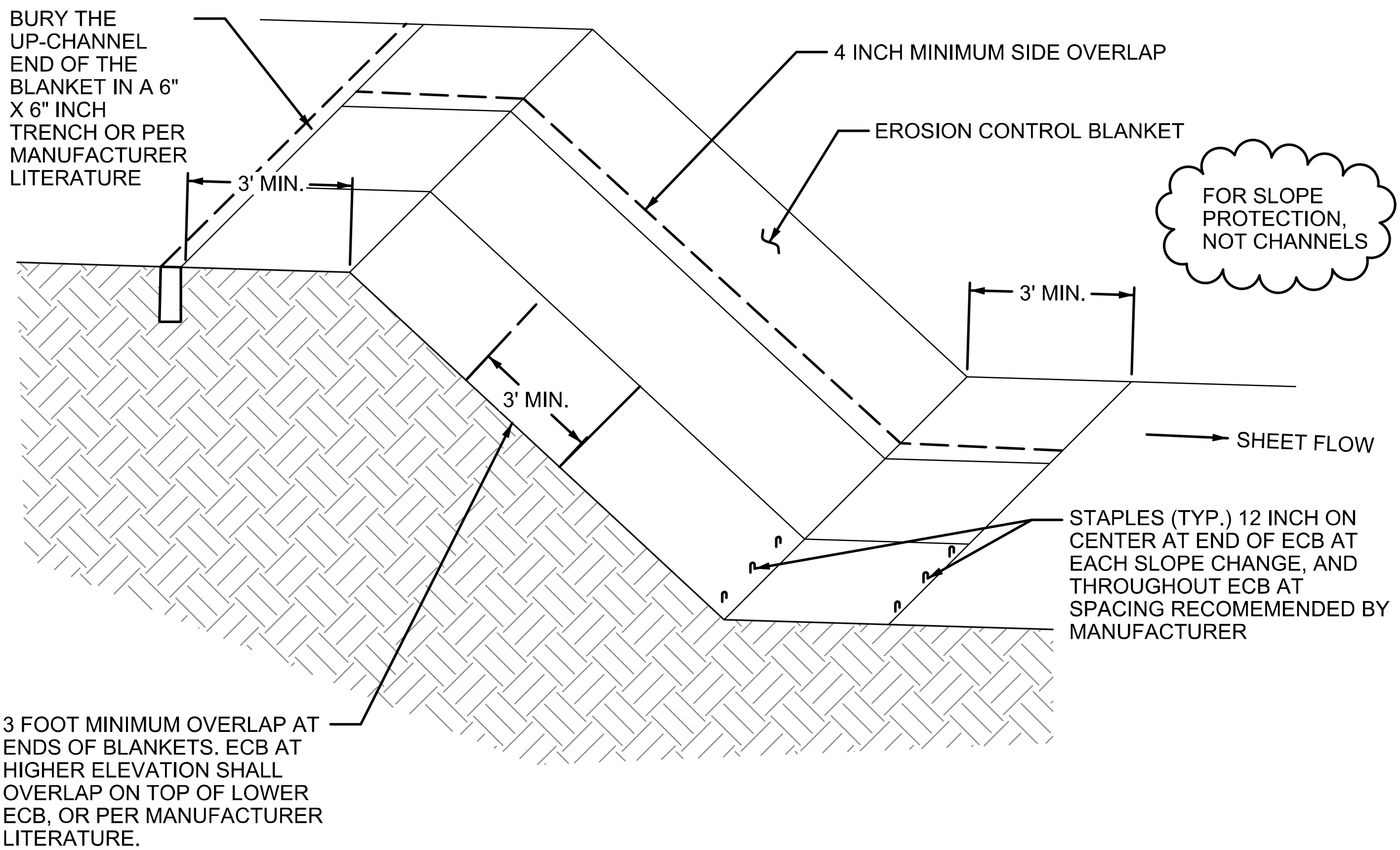
2. STONE SHALL BE WELL GRADED WITH SIZE RANGE FROM 1 1/2 TO 3 1/2 INCHES IN DIAMETER DEPENDING ON EXPECTED FLOWS.

3. THE CHECK DAM SHALL BE INSPECTED AS SPECIFIED IN THE SWPPP AND SHALL BE REPLACED WHEN THE STRUCTURE CEASES TO FUNCTION AS INTENDED DUE TO SILT ACCUMULATION AMONG THE ROCKS, WASHOUT, CONSTRUCTION TRAFFIC DAMAGE, ETC.

4. WHEN SILT REACHES A DEPTH EQUAL TO ONE-THIRD OF THE HEIGHT OF THE CHECK DAM OR ONE FOOT, WHICHEVER IS LESS, THE SILT SHALL BE REMOVED AND DISPOSED OF PROPERLY.

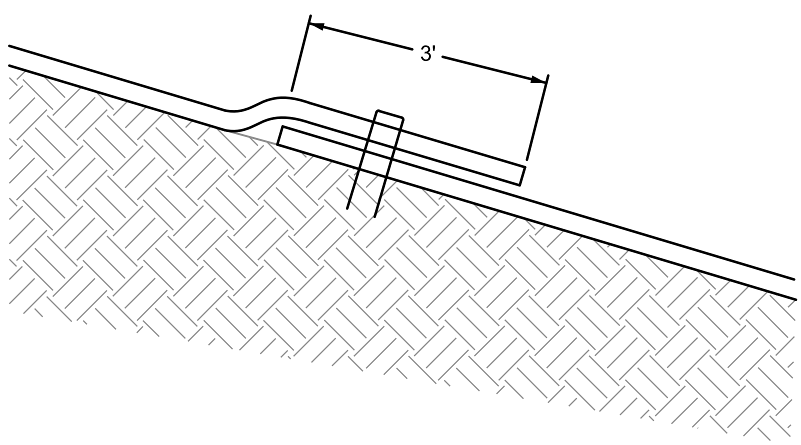
5. WHEN THE SITE HAS ACHIEVED FINAL STABILIZATION OR ANOTHER EROSION OR SEDIMENT CONTROL DEVICE IS EMPLOYED, THE CHECK DAM AND ACCUMULATED SILT SHALL BE REMOVED AND DISPOSED OF IN AN APPROVED MANNER.

FIGURE 2.1 NOTES ON ROCK CHECK DAM (2 OF 2)



ECB ISOMETRIC PLAN VIEW

N.T.S.



ECB OVERLAP EXAMPLE

N.T.S.

FIGURE 2.7 STANDARD CONSTRUCTION DETAIL - TEMPORARY EROSION CONTROL BLANKETS (1 OF 2)

EROSION CONTROL BLANKETS GENERAL NOTES:

1. SEE NCTCOG STANDARD SPECIFICATIONS (2017) SECTION 202.15.

2. PRIOR TO THE INSTALLATION OF ANY EROSION CONTROL BLANKETS, ALL ROCKS, DIRT CLODS, STUMPS, ROOTS, TRASH AND ANY OTHER OBSTRUCTIONS THAT WOULD PREVENT THE BLANKET FROM LYING IN DIRECT CONTACT WITH THE SOIL SHALL BE REMOVED. ANCHOR TRENCHING SHALL BE LOCATED ALONG THE ENTIRE PERIMETER OF THE INSTALLATION AREA, EXCEPT FOR SMALL AREAS WITH LESS THAN 2% SLOPE.

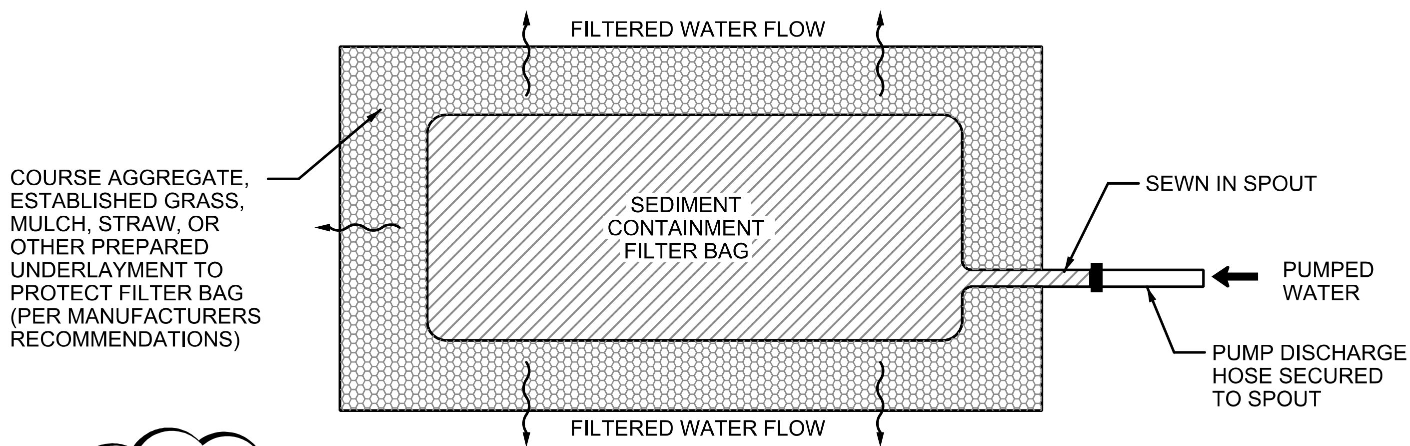
3. INSTALLATION AND ANCHORING SHALL CONFORM TO THE RECOMMENDATIONS SHOWN WITHIN THE MANUFACTURER'S PUBLISHED LITERATURE FOR THE APPROVED EROSION CONTROL BLANKET. PARTICULAR ATTENTION MUST BE PAID TO JOINTS AND OVERLAPPING MATERIAL.

4. IN ABSENCE OF MANUFACTURE'S LITERATURE, A MINIMUM 11-GUAGE WIRE STAPLES, 6-INCHES IN LENGTH AND 1-INCH WIDTH WILL BE USED.

5. AFTER APPROPRIATE INSTALLATION, THE BLANKETS SHOULD BE CHECKED FOR UNIFORM CONTACT WITH THE SOIL, SECURITY OF THE LAP JOINTS, AND FLUSHNESS OF THE STAPLES WITH THE GROUND.

6. INSPECTION SHALL BE AS SPECIFIED IN THE SWPPP.

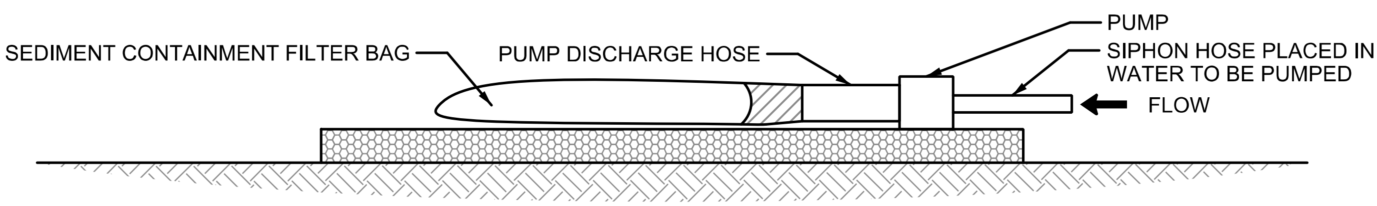
FIGURE 2.7 NOTES ON TEMPORARY EROSION CONTROL BLANKETS (2 OF 2)



SEDIMENT FILTER BAG PLAN VIEW

N.T.S.

SHOW LOCATIONS OF FILTER BAGS ON THE DRAWINGS.



SEDIMENT FILTER BAG PROFILE

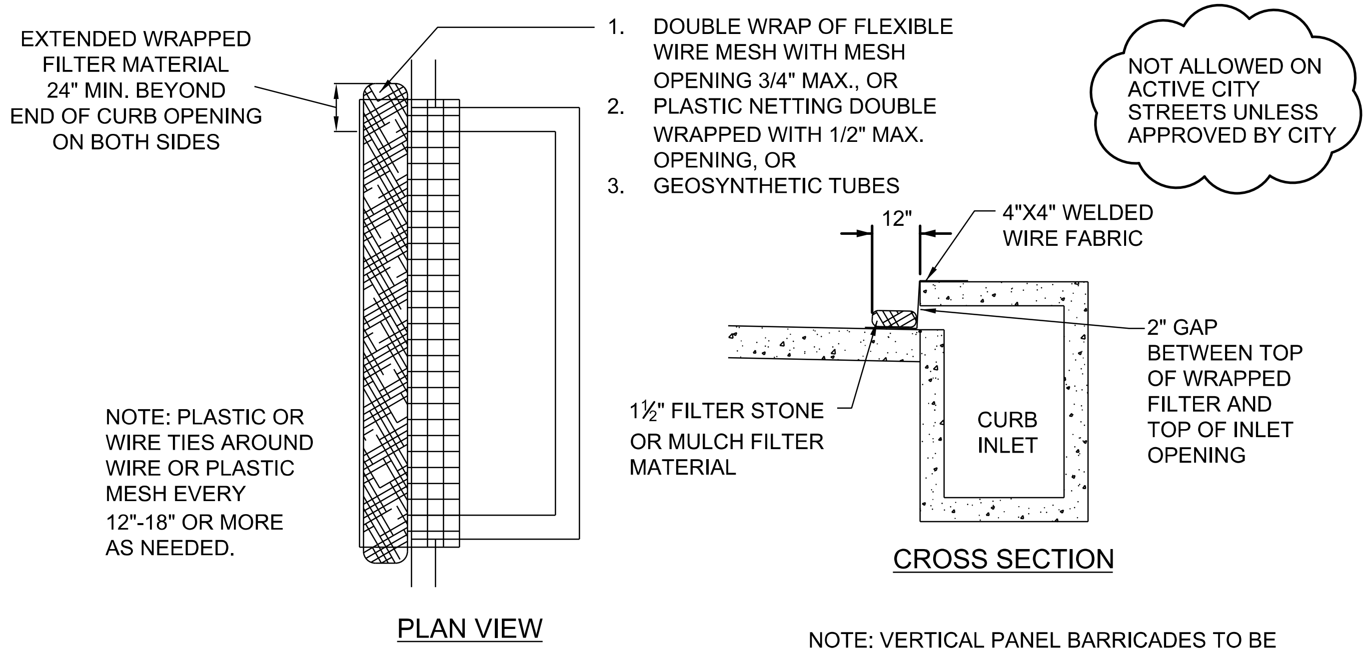
N.T.S.

DEWATERING CONTROL GENERAL NOTES:

1. THE BAG SHOULD BE A NON-WOVEN, NEEDLE-PUNCHED, GEOTEXTILE THAT MEETS ASTM CRITERIA-D4632 , D4833, D-3786, D-4355, D-4491, AND D-4751.

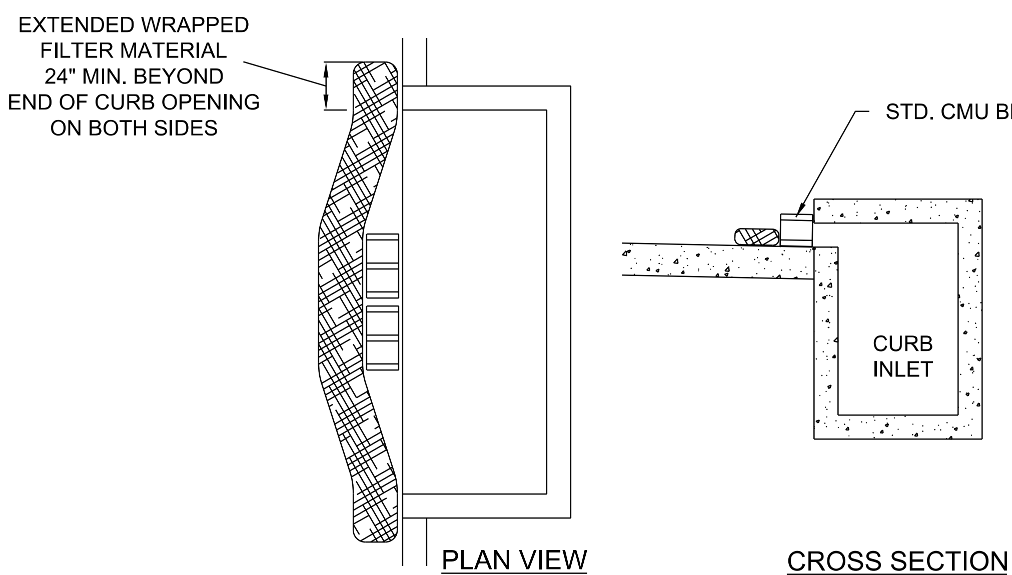
2. CAPACITY, INSTALLATION, MAINTENANCE , AND REMOVAL OF BAGS AND PUMPS SHOULD CONFORM TO PUBLISHED MANUFACTURER LITERATURE.

FIGURE 3.4 STANDARD CONSTRUCTION DETAIL - DEWATERING CONTROLS



NOTE: VERTICAL PANEL BARRICADES TO BE PLACED WHEN LOCATED ON AN ACTIVE STREET.

TYPE A CURB INLET PROTECTION

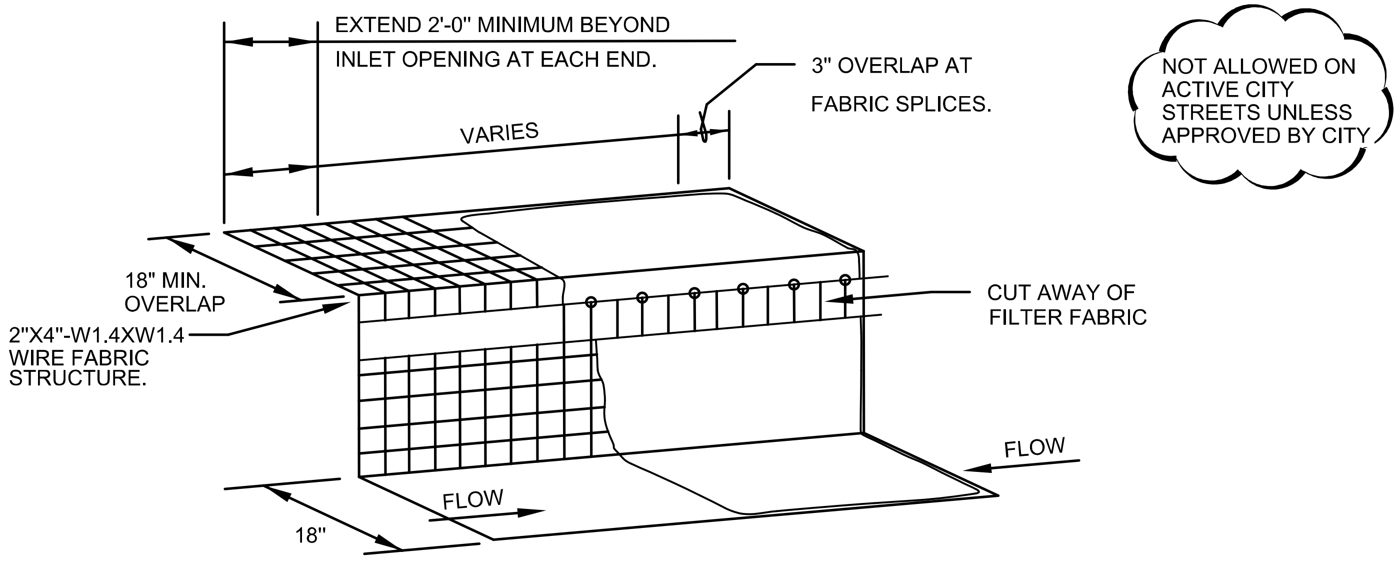


CURB OPENING	MIN. NO. BLOCKS
4'-6'	1
8'-10'	2
12'-14'	3
16'-20'	4

ALTERNATIVE FORM FOR TYPE A CURB INLET PROTECTION

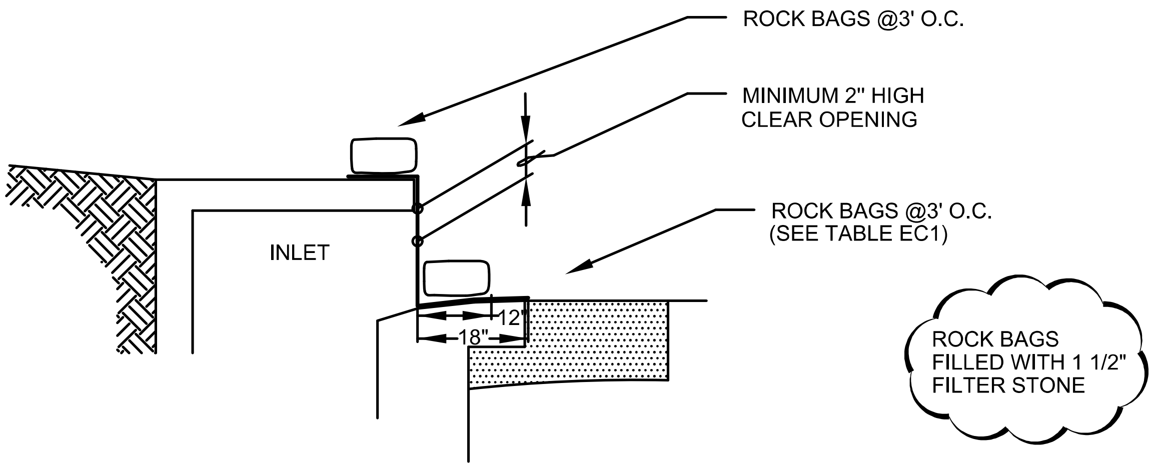
NOTE: SEE NCTCOG STANDARD SPECIFICATIONS (2017), SECTION 202.14 AND 202.18

FIGURE 3.6 STANDARD CONSTRUCTION DETAIL - FILTER TUBE CURB INLET PROTECTION



HOG WIRE WEIR CURB INLET PROTECTION ISOMETRIC VIEW

N.T.S.



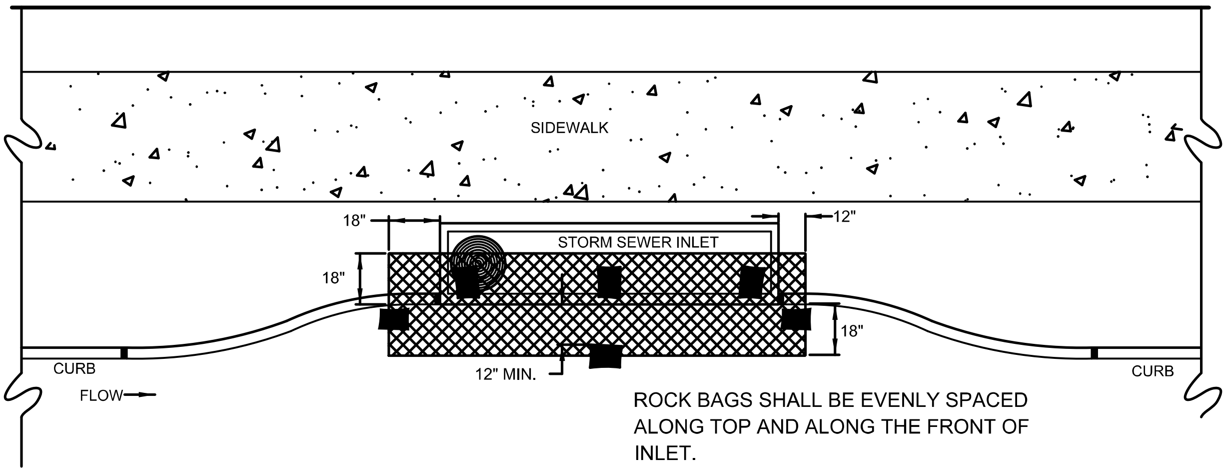
HOG WIRE WEIR CURB INLET PROTECTION CROSS SECTION

N.T.S.

NOTE: THIS CONTROL WILL DECREASE THE CAPACITY OF THE INLET. IT SHALL ONLY BE USED WHEN AN ENGINEER HAS DETERMINED THERE IS ADEQUATE STORAGE OR POSITIVE OVERFLOW.

REFERENCE: NCTCOG STANDARD SPECIFICATIONS (2017), SECTION 202.14

FIGURE 3.7 STANDARD CONSTRUCTION DETAIL -
HOG WIRE WEIR CURB INLET PROTECTION (1 OF 2)



HOG WIRE WEIR CURB INLET PROTECTION PLAN VIEW

N.T.S.

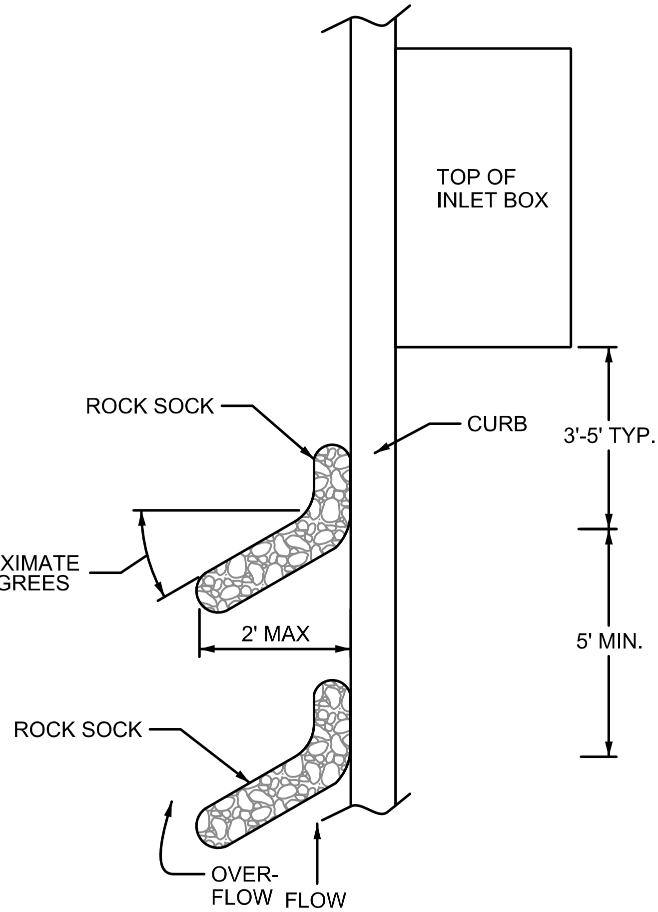
TABLE EC1

INLET OPENING	MINIMUM NUMBER OF ROCK BAGS	
	TOP	FRONT
5'	2	3
10'	3	3
15'	3	4
20'	4	4

NOTES:

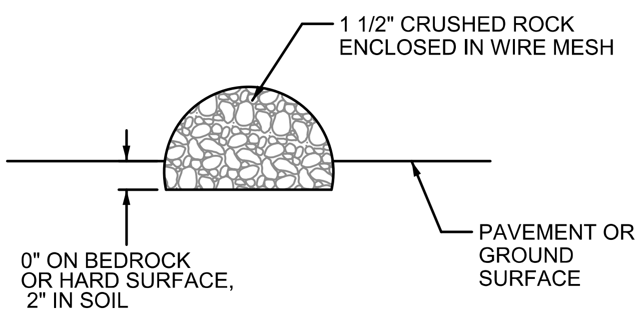
1. A SECTION OF FILTER FABRIC SHALL BE REMOVED AS SHOWN ON THIS DETAIL TO PROVIDE A 2" MINIMUM CLEAR OPENING. FABRIC MUST BE SECURED TO WIRE BACKING WITH CLIPS OR HOG RINGS AT THIS LOCATION.
2. INSPECTION SHALL BE MADE BY THE CONTRACTOR AND SILT ACCUMULATION MUST BE REMOVED WHEN DEPTH REACHES 2".
3. INLET PROTECTIONS SHALL BE REMOVED AS SOON AS THE SOURCE OF SEDIMENT IS STABILIZED.

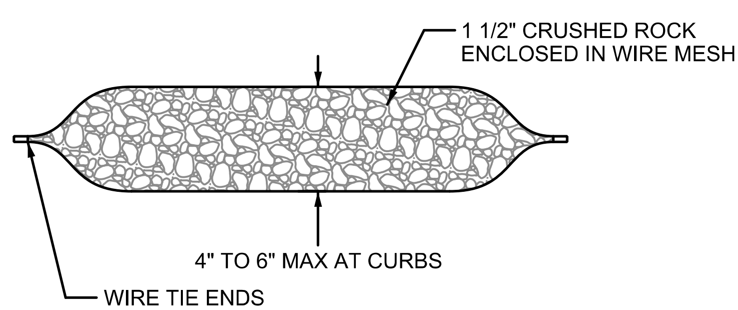
FIGURE 3.7 STANDARD CONSTRUCTION DETAIL - HOG WIRE WEIR CURB INLET PROTECTION (2 OF 2)



CURB ROCK SOCK ON-GRADE CURB INLET PROTECTION DETAIL

N.T.S.





ROCK SOCK SECTION

N.T.S.

ROCK SOCK PLAN

N.T.S.

FIGURE 3.9 STANDARD CONSTRUCTION DETAIL -
CURB ROCK SOCK ON-GRADE CURB INLET PROTECTION (1 OF 2)

CURB ROCK SOCK ON-GRADE CURB INLET PROTECTION GENERAL NOTES:

1. THIS DETAIL IS INTENDED FOR USE WITH ON-GRADE INLETS (NOT A LOW POINT) TO TRAP SEDIMENT.
2. DO NOT INSTALL ON INLETS WHERE THE ROCK SOCKS WOULD EXTEND INTO AN ACTIVE TRAVEL LANE.
3. ROCK SOCKS MAY BE USED ON PAVED OR UNPAVED SURFACES.
4. MAXIMUM ROCK SOCK DIAMETER 4" TO 6".
5. MINIMUM OF 2 CURB ROCK SOCKS.

FIGURE 3.9 STANDARD CONSTRUCTION DETAIL -
CURB ROCK SOCK ON-GRADE CURB INLET PROTECTION (2 OF 2)

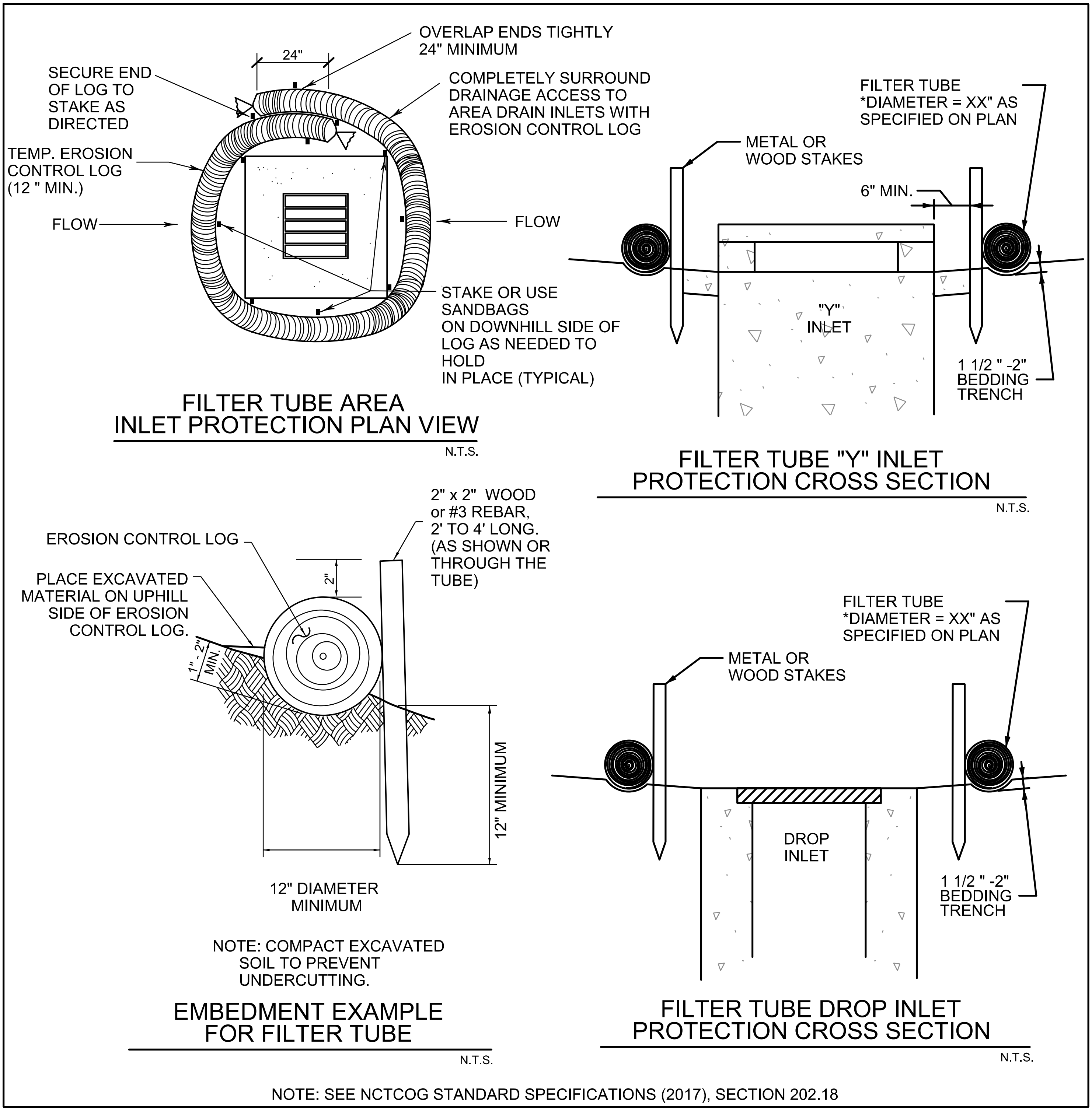
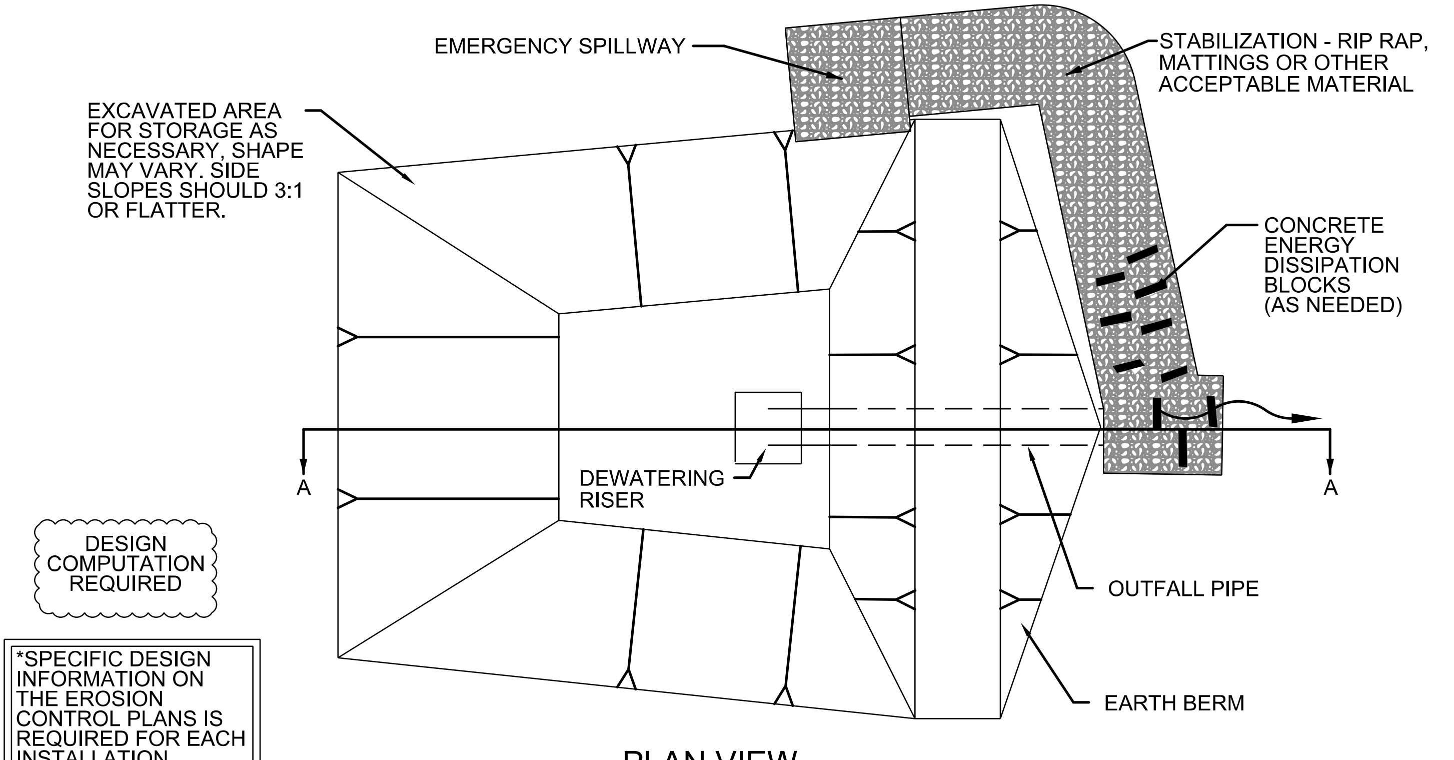
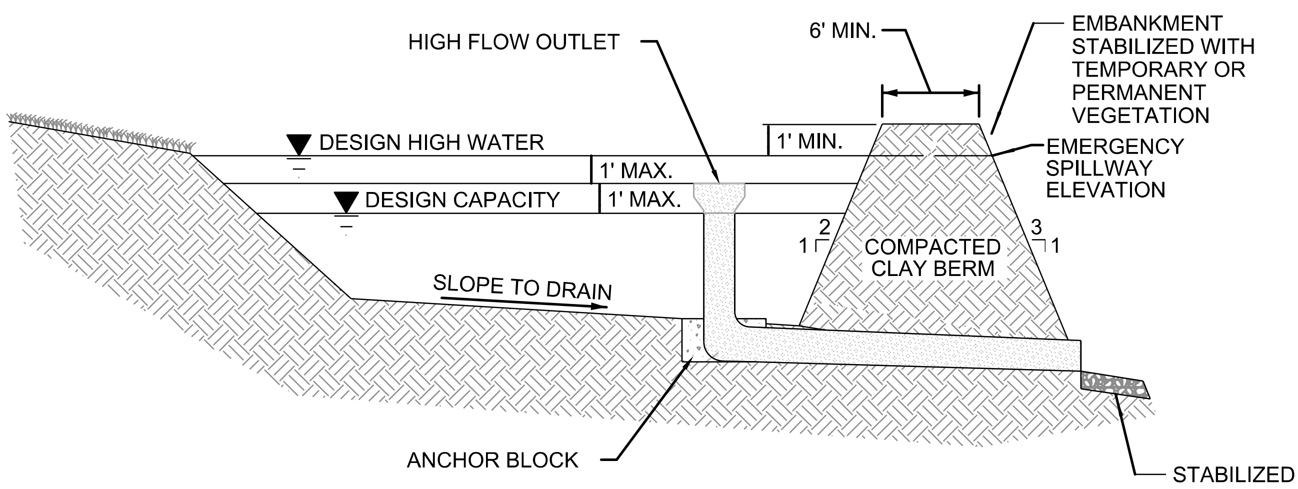


FIGURE 3.13 STANDARD CONSTRUCTION DETAIL - FILTER TUBE AREA INLET PROTECTION



PLAN VIEW

N.T.S.

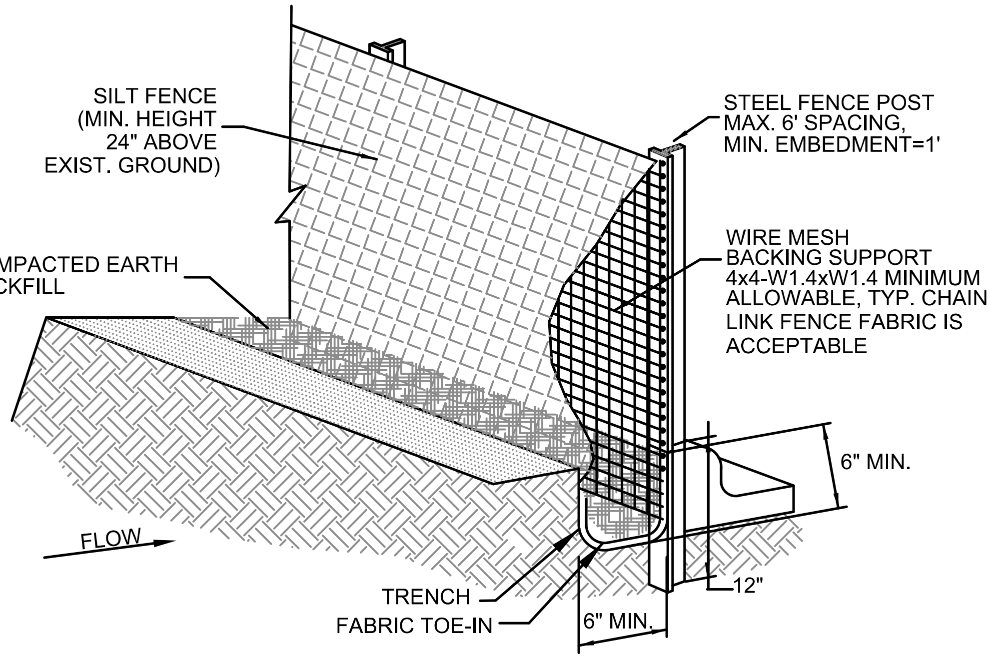


CROSS SECTION A-A

N.T.S.

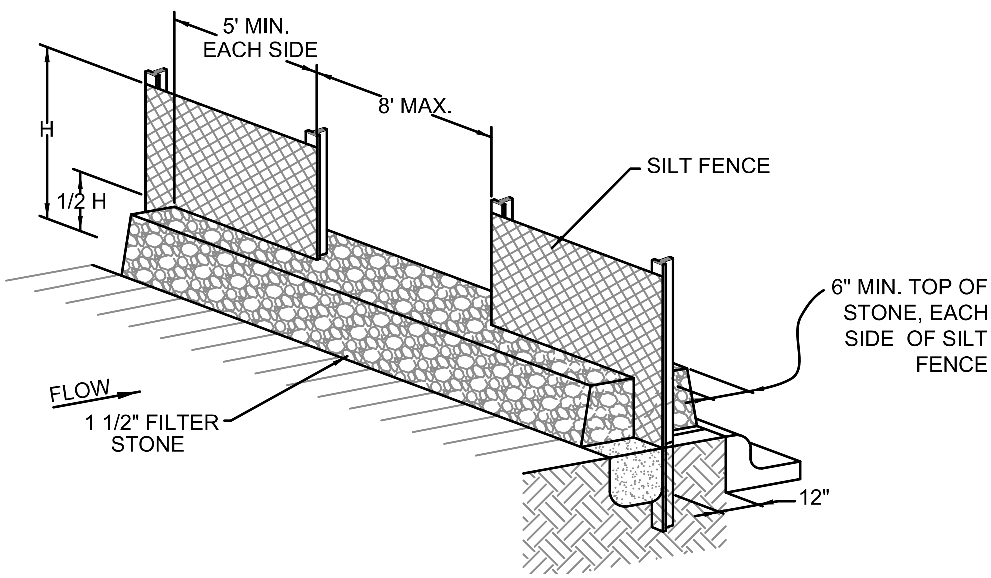
REFERENCE: NCTCOG iSWM CRITERIA MANUAL FOR SITE DEVELOPMENT AND CONSTRUCTION, SECTION 3.9 SEDIMENT BASIN.

FIGURE 3.20 STANDARD CONSTRUCTION DETAIL - SEDIMENT BASIN WITH OVERFLOW RISER



ISOMETRIC PLAN VIEW

N.T.S.



SILT FENCE OVERFLOW STRUCTURE

N.T.S.

FIGURE 3.28 STANDARD CONSTRUCTION DETAIL - FOR SILT FENCE (1 OF 2)

SILT FENCE GENERAL NOTES:

1. DESIGN SHALL SHOW ON THE DRAWINGS THE LOCATIONS WHERE OVERFLOW STRUCTURES SHALL BE INSTALLED. OVERFLOW STRUCTURES ARE REQUIRED AT ALL LOW POINTS AND AT A SPACING OF APPROXIMATELY 300 FEET WHERE NO LOW POINT IS APPARENT.

2. DESIGNER SHALL SHOW ON THE DRAWINGS THE LOCATIONS WHERE SILT FENCE IS TO BE TURNED UPSLOPE AT THE ENDS. UPSLOPE LENGTHS SHALL BE A MINIMUM OF 10 FEET.

3. POST WHICH SUPPORT THE SILT FENCE SHALL BE INSTALLED ON A SLIGHT ANGLE TOWARD THE ANTICIPATED RUNOFF SOURCE. POST MUST BE EMBEDDED A MINIMUM OF ONE FOOT.

4. THE TOE OF THE SILT FENCE SHALL BE TRENCHED IN WITH A SPADE OR MECHANICAL TRENCHER, SO THAT THE DOWNSLOPE FACE OF THE TRENCH IS FLAT AND PERPENDICULAR TO THE LINE OF FLOW.

5. THE TRENCH MUST BE A MINIMUM OF 6 INCHES DEEP AND 6 INCHES WIDE TO ALLOW FOR THE SILT FENCE FABRIC TO BE LAID IN THE GROUND AND BACKFILLED WITH COMPACTED MATERIAL.

6. SILT FENCE SHOULD BE SECURELY FASTENED TO EACH SUPPORT POST OR TO WIRE BACKING, WHICH IN TURN IS ATTACHED TO THE FENCE POST. THERE SHALL BE A 3 FOOT OVERLAP, SECURELY FASTENED WHERE ENDS OF FABRIC MEET.

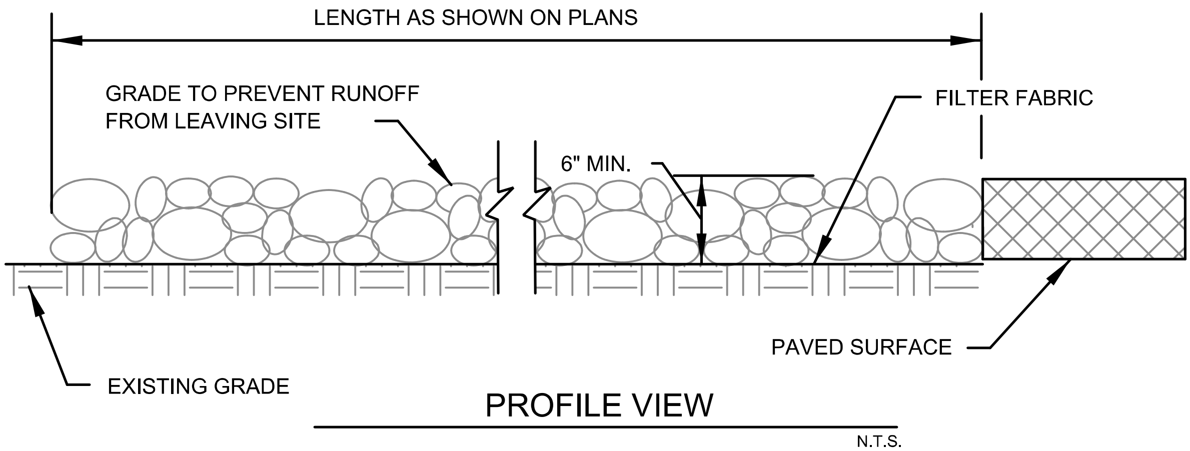
7. INSPECTION SHALL BE AS SPECIFIED IN THE SWPPP. REPAIR OR REPLACEMENT SHALL BE MADE PROMPTLY AS NEEDED.

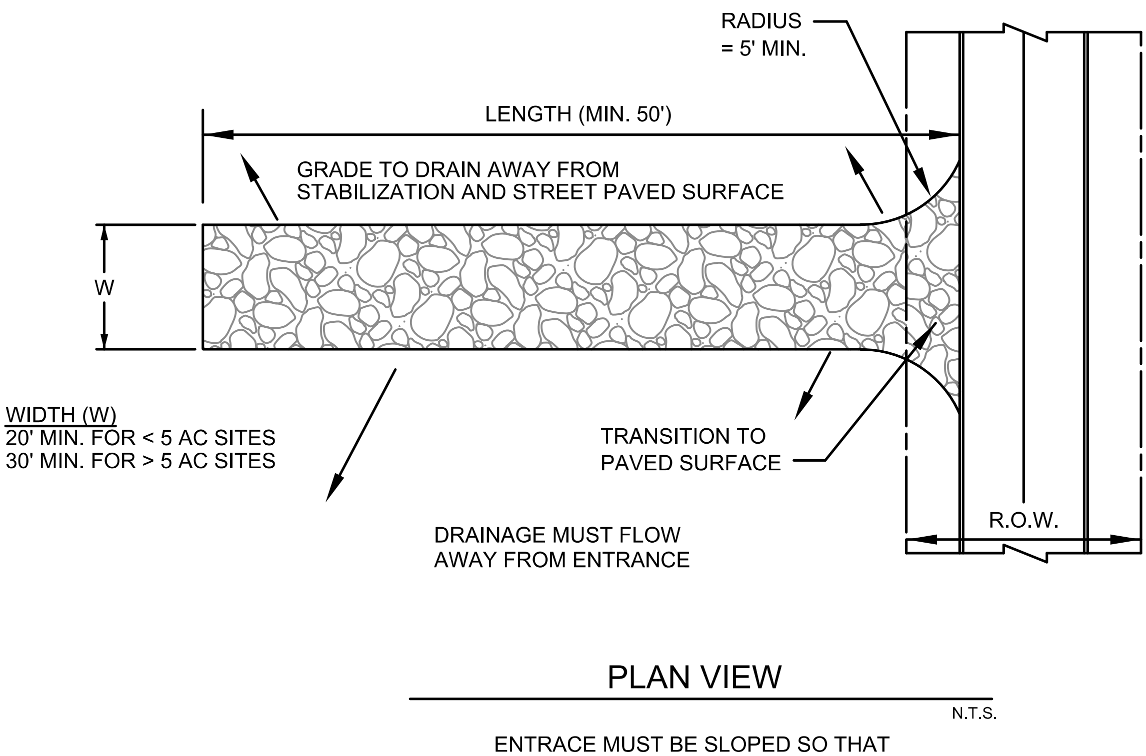
8. SILT FENCE SHALL BE REMOVED WHEN FINAL STABILIZATION IS ACHIEVED OR ANOTHER EROSION OR SEDIMENT CONTROL DEVICE IS EMPLOYED.

9. ACCUMULATED SILT SHALL BE REMOVED WHEN IT REACHES A DEPTH OF HALF THE HEIGHT OF THE FENCE. THE SILT SHALL BE DISPOSED OF AT AN APPROVED SITE AND IN SUCH A MANNER AS TO NOT CONTRIBUTE TO ADDITIONAL SILTATION.

10. SEE NCTCOG STANDARD SPECIFICATIONS (2017), SECTION 202.5

FIGURE 3.28 NOTES FOR SILT FENCE (2 OF 2)





ENTRACE MUST BE SLOPED SO THAT STORM WATER IS NOT ALLOWED TO LEAVE THE SITE AND ENTER ROADWAYS

FIGURE 3.29 STANDARD CONSTRUCTION DETAIL - STABILIZED CONSTRUCTION EXIT (1 OF 2)

STABILIZED CONSTRUCTION ENTRANCE GENERAL NOTES:

1. SEE NCTCOG STANDARD SPECIFICATIONS (2017), SECTION 202.11

2. THE THICKNESS SHALL NOT BE LESS THAN 6 INCHES.

3. STONE SHALL BE 3 TO 5 INCH DIAMETER COURSE AGGREGATE, NO CRUSHED PORTLAND CEMENT CONCRETE ALLOWED.

4. LENGTH SHALL BE SHOWN ON PLANS, WITH A MINIMUM LENGTH OF 50 FEET.

5. THE WIDTH SHALL BE NO LESS THAN 20' FOR SITES LESS THAN 5 AC, AND 30' FOR SITES GREATER THAN 5 AC, AT ALL POINTS OF INGRESS OR EGRESS.

6. WHEN NECESSARY, VEHICLES SHALL BE CLEANED TO REMOVE SEDIMENT PRIOR TO ENTRANCE ONTO A PUBLIC ROADWAY. WHEN WASHING IS REQUIRED, IT SHALL BE DONE ON AN AREA STABILIZED WITH CRUSHED STONE WITH DRAINAGE FLOWING AWAY FROM BOTH THE STREET AND THE STABILIZED ENTRANCE. ALL SEDIMENT SHALL BE PREVENTED FROM ENTERING ANY STORM DRAIN, DITCH OR WATERCOURSE USING APPROVED METHODS.

7. THE ENTRANCE SHALL BE MAINTAINED IN A CONDITION WHICH WILL PREVENT TRACKING OR FLOWING OF SEDIMENT ONTO PAVED SURFACES. THIS MAY REQUIRE PERIODIC TOP DRESSING WITH ADDITIONAL STONE AS CONDITIONS DEMAND. ALL SEDIMENT SPILLED, DROPPED, WASHED, OR TRACKED ONTO PAVED SURFACES MUST BE REMOVED IMMEDIATELY.

8. THE ENTRANCE MUST BE PROPERLY GRADED OR INCORPORATE A DRAINAGE SWALE TO PREVENT RUNOFF FROM LEAVING THE CONSTRUCTION SITE.

9. INSPECTION SHALL BE SPECIFIED IN THE SWPPP.

FIGURE 3.29 NOTES FOR STABILIZED CONSTRUCTION EXIT (2 OF 2)

Landscape:

1.0 Landscape and Aesthetics Guidance

Table of Contents

List of Tables

List of Figures

1.0 Landscape and Aesthetics Guidance

1.1 Introduction

Landscaping is a critical element in the design of stormwater facilities for water quantity and quality management, serving both functional and aesthetic purposes. Plants and vegetation perform a number of functions in stormwater 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 stormwater facilities

The purpose of this section is to provide guidance on landscaping and plant selection for stormwater facilities and structural controls, as well as provide an overview on developing aesthetically-pleasing stormwater facilities. This section is divided into the following subsections:

- Section 1.2 covers general landscaping guidance that should be considered when landscaping any stormwater facility.

- Section 1.3 discusses the physical site factors and considerations involved in selecting plant material for stormwater facility landscaping.

- Section 1.4 includes key factors to consider in selecting plant material for stormwater landscaping are reviewed, including hardiness, physiographic regions, inundation tolerance, and other factors.

- Section 1.5 outlines more specific guidance on landscaping criteria and plant selection for individual structural stormwater control designs, including:

  - Stormwater Ponds and Wetlands
  - Bioretention Areas
  - Infiltration Trench and Surface Sand Filter Facilities
  - Enhanced Swales and Grass Channels
  - Filter Strips and Stream Buffers
  - Green Roofs

- Section 1.6 contains a detailed plant list of trees and shrubs that may be used when preparing a vegetation and landscaping planting plan for a stormwater facility.

Review local landscape ordinance requirements before developing a landscape plan.

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 the North Central Texas SmartScape website at www.txsmartscape.com. Using native and adapted plants reduces the need for excessive fertilizer in runoff, which can lead to “overgrowth” of submerged plants.

1.2 General Landscaping Guidance

Below are general guidelines that should be followed in the landscaping of any stormwater 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:

- Review the local landscape ordinance requirements.
- Take into account site characteristics and plant selection guidelines (see *Sections 1.3 and 1.4*, respectively) when selecting plants for stormwater facilities.
- Consider how plant characteristics will affect the landscape and the performance of a structural stormwater 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:
  - Stormwater management facilities to help educate the public
  - Wildflower areas to designate limits of mowing
  - Preserving existing natural vegetation

1.3 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.

1.4 Plant Selection for Stormwater Facilities

1.4.1 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 section 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.

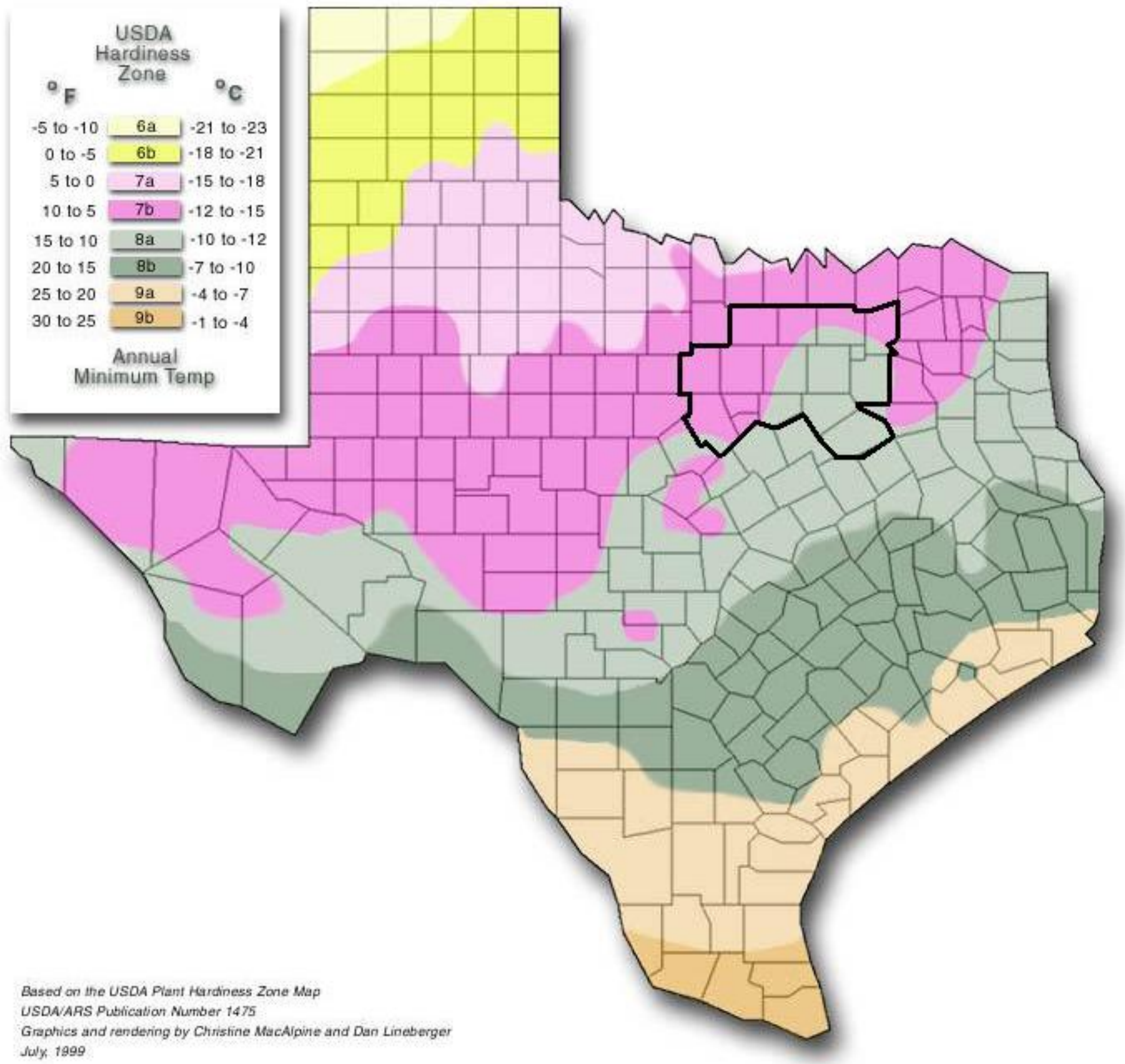
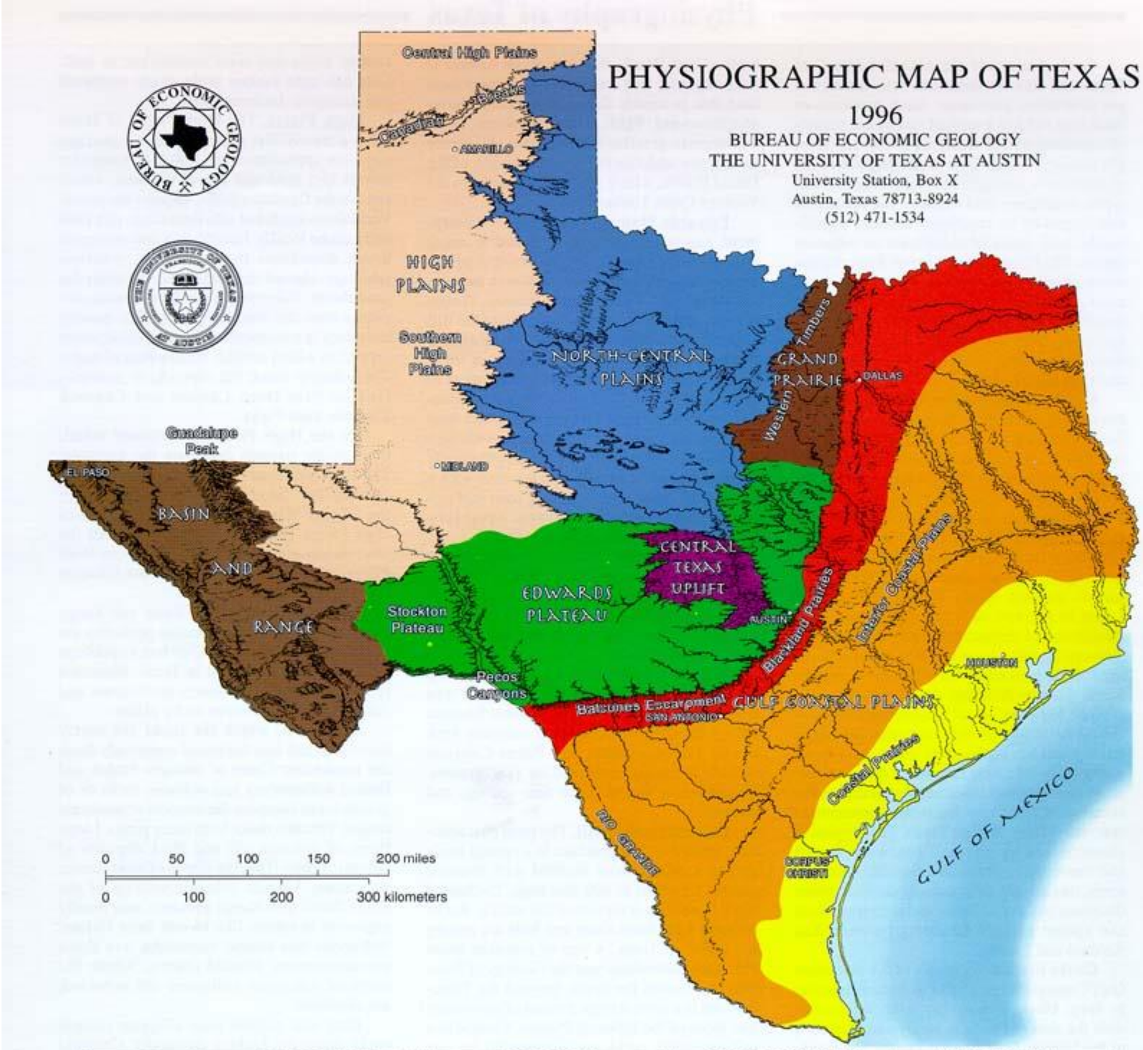


Figure 1.1 USDA Plant Hardiness Zones in Texas

1.4.2 Physiographic Provinces

There are three physiographic provinces in Texas that describe distinct geographic regions in the state with similar physical and environmental conditions (Figure 1.2). 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 stormwater 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 1.2 Physiographic map of Texas

<u>Gulf Coastal Plains.</u> 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 stormwater 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.

1.4.3 Other Considerations in Plant Selection

Use or Function

In selecting plants, consideration must be given to their desired function in the stormwater 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 stormwater 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 stormwater 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 stormwater 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. 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 the North Central Texas SmartScape website at www.txsmartscape.com.

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 stormwater management facility. Since many stormwater facilities are adjacent to residential areas, the objectives of the stormwater 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 stormwater 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 stormwater management facility it is necessary to determine what type of hydrologic zones will be created within the facility.

The six zones shown in Table 1.1 in the next section describe the different conditions encountered in stormwater 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 stormwater functions required of the plant and the desired landscape effect.

1.5 Specific Landscaping Criteria for Structural Stormwater Controls

1.5.1 Stormwater Ponds and Wetlands

Stormwater ponds and wetlands are engineered basins and wetland areas designed to control and treat stormwater runoff. Aquatic vegetation plays an important role in pollutant removal in both stormwater 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 stormwater pond or wetland, there are various hydrologic zones as shown in Table 1.1 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 1.1 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 stormwater 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 stormwater wetlands. Zone 2 also coincides with the aquatic bench found in stormwater 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 stormwater wetlands and on the aquatic benches of stormwater 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 stormwater 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 1.2 provides a list of selected wetland plants for stormwater 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 1.2 Wetland Plants (Herbaceous Species) for Stormwater 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
Eustoma grandiflora	Texas Bluebells	4
Helianthus angustifolius	Swamp Sunflower	2
Helianthus maximiliani	Maximilian Sunflower	3,4,5,6
Hibiscus laevis	Halberdleaf Hibiscus	2,3

Table 1.2 Wetland Plants (Herbaceous Species) for Stormwater Facilities		
Scientific Name	**Common Name**	**Hydrologic Zone**
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
Scirpus americanus	Three-square	2
Scirpus californicus	Giant Bulrush	2
Scirpus validus	Softstem Bulrush	2,3

Table 1.2 Wetland Plants (Herbaceous Species) for Stormwater Facilities		
Scientific Name	**Common Name**	**Hydrologic Zone**
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 1.3 Legend of Hydrologic Zones Around Stormwater Facilities

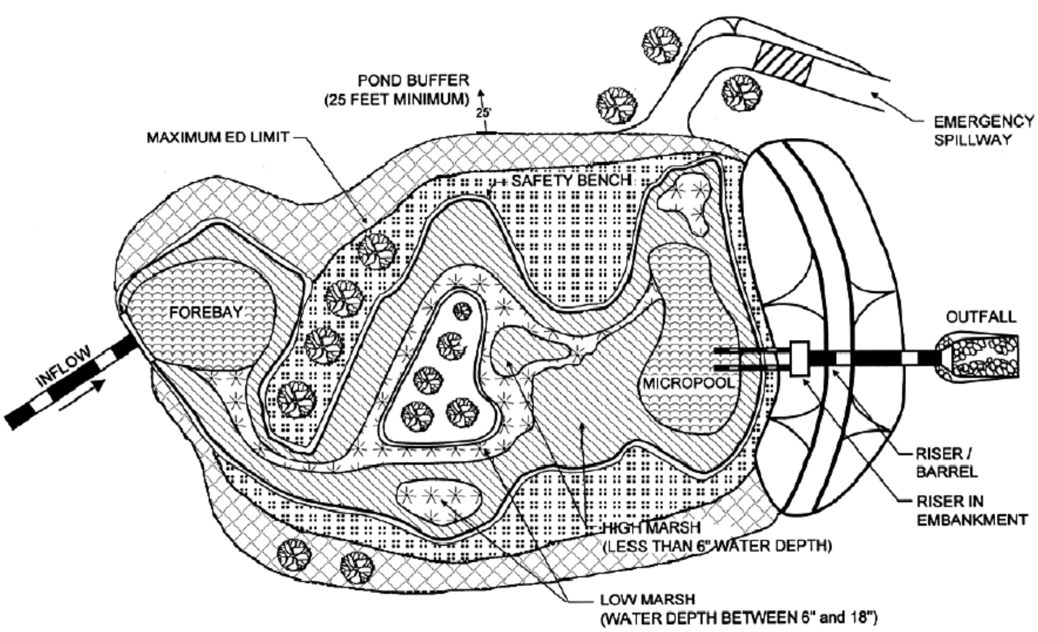


Figure 1.4 Plan View of Hydrologic Zones around Stormwater Wet ED Pond

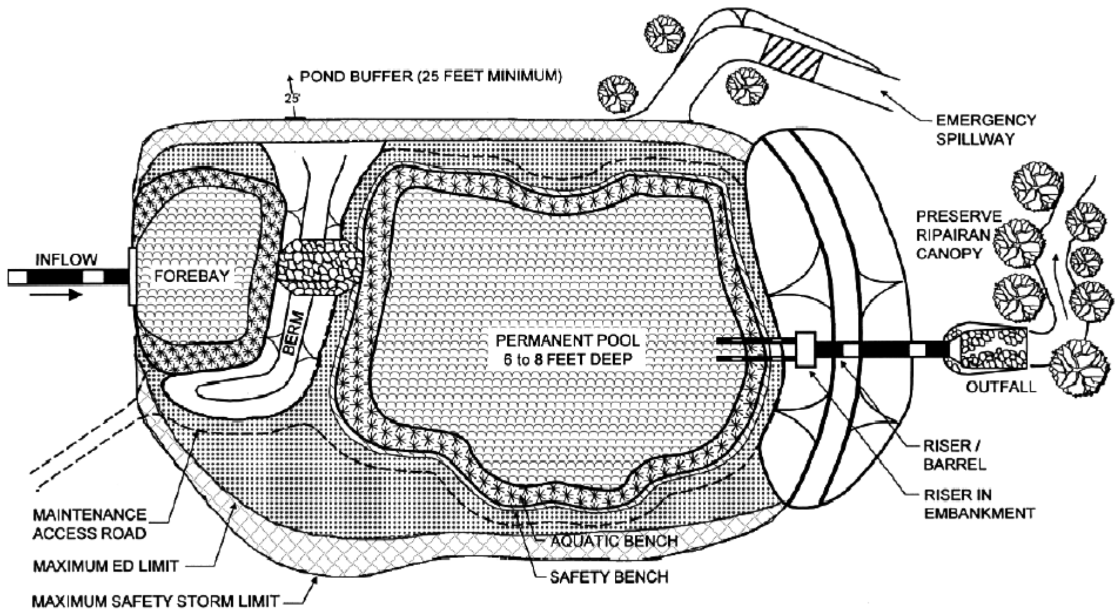


Figure 1.5 Plan View of Hydrologic Zones around Stormwater ED Shallow Wetland

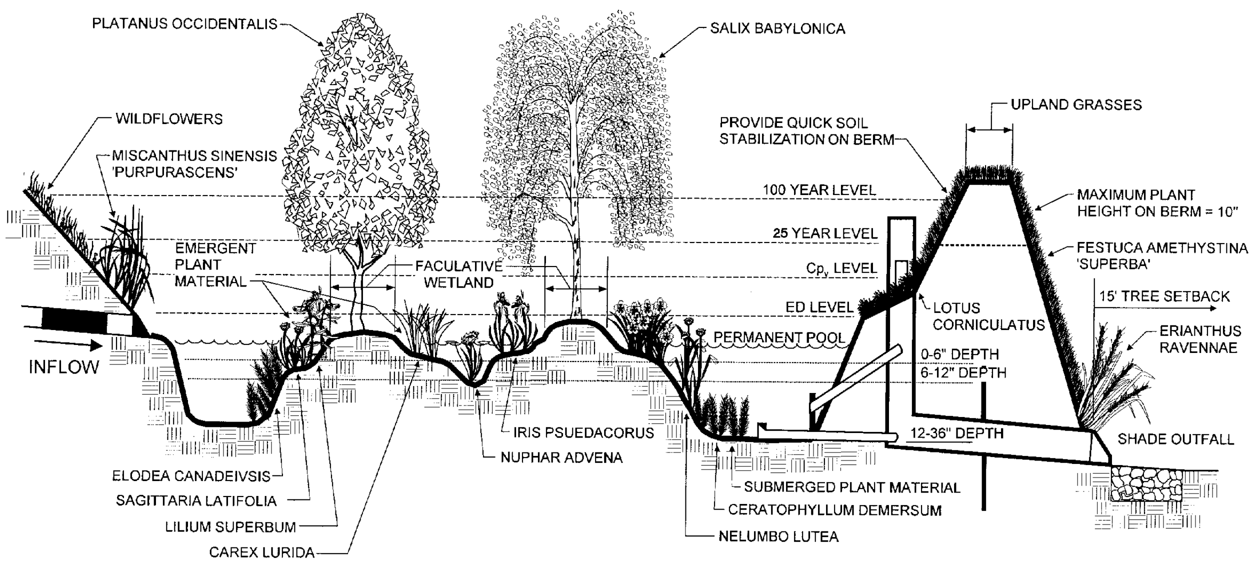


Figure 1.6 Section of Typical Shallow ED Wetland

1.5.2 Bioretention Areas

Bioretention areas are structural stormwater 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 1.3.

Table 1.3 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 stormwater 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 1.7). 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 1.4. More potential bioretention species can be found in the wetland plant list in *Section 1.6*.

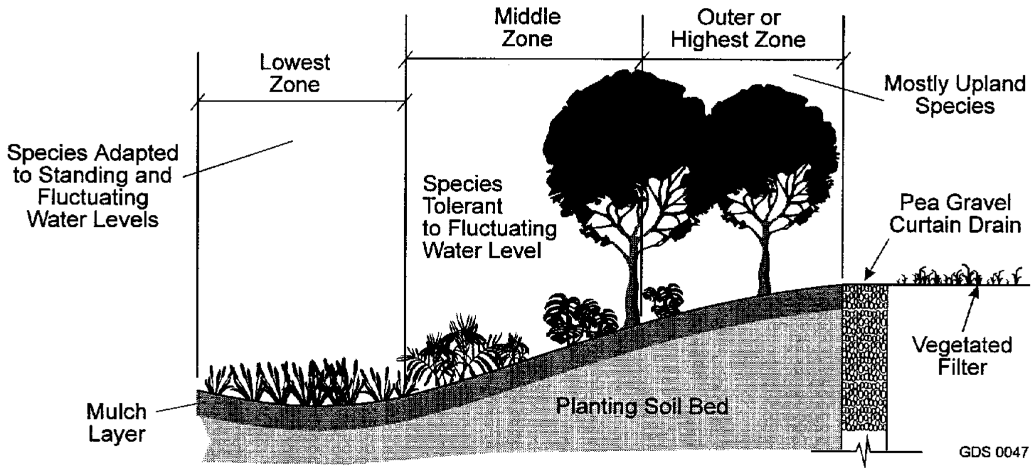


Figure 1.7 Planting Zones for Bioretention Facilities

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 1.5 presents some typical issues for planting specifications. Figure 1.8 shows an example of a sample planting plan for a bioretention area.

Table 1.4 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 1.5 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.

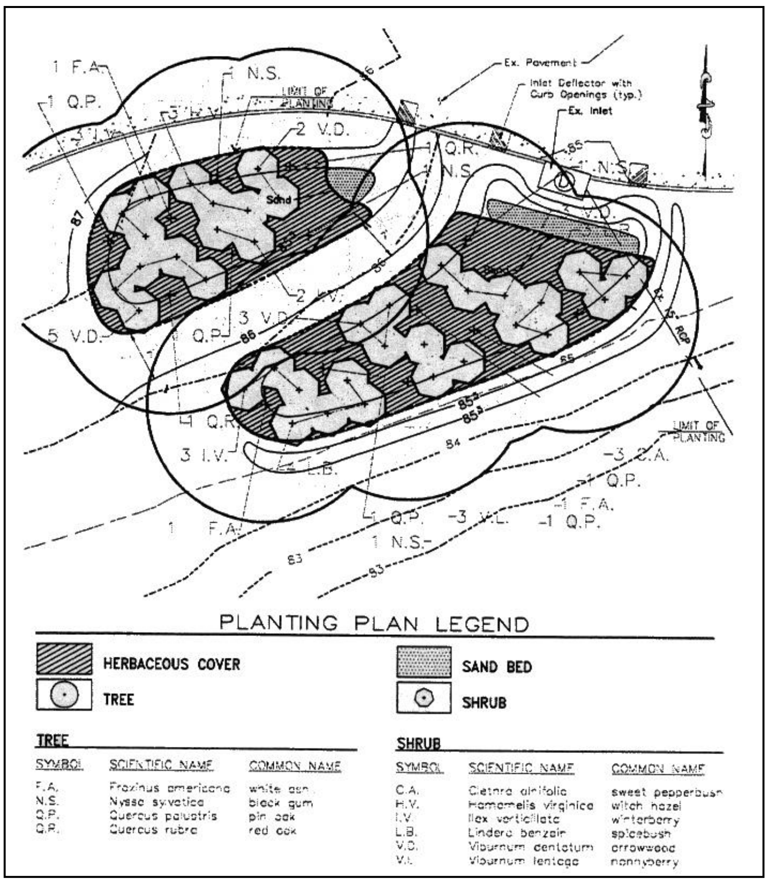


Figure 1.8 Sample Bioretention Area Planting Plan
(Source: VDCR, 1999)

1.5.3 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.

1.5.4 Enhanced Swales, Grass Channels and Filter Strips

Table 1.6 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 *Section 1.5.1*). Select plant material capable of salt tolerance in areas that may include high salt levels.

Table 1.6 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
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

1.5.5 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.

1.6 Trees and Shrubs for Stormwater Facilities

The following pages present a detailed list of wetland trees and shrubs that may be used for stormwater management facilities such as stormwater ponds, stormwater wetlands and bioretention areas. (Source: Garber and Moorhead, 1999)

Table 1.7 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	

Table 1.7 continued

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	

Table 1.7 continued

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 1.8 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 1.8 continued

<u>**Species**</u>	<u>**Water Level**</u>	<u>**Seedling Survival***</u>	<u>**Comments**</u>
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

1.7 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.

Texas SmartScape, www.txsmartscape.com, North Central Texas Council of Governments

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

Transportation *integrated* Stormwater Management (TriSWM) Appendix

Table of Contents

List of Tables

List of Figures

1.0 Overview of TriSWM Appendix

1.1 Introduction

The TriSWM Appendix has been developed as an appendix to the iSWM Criteria Manual for Site Development and Construction for use by cities, counties, and transportation agencies in the planning and design of stormwater management systems for public streets, roads, and highways. The purpose of this Appendix is to provide design guidance and a framework for incorporating effective and environmentally sustainable stormwater management into the project development and construction processes and to encourage a greater regional uniformity in developing plans for stormwater management systems that meet the following goals:

- Control 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 stormwater runoff to protect water quality and assist communities in meeting regulatory requirements.

The table below indicates the chapters or sections of the iSWM Criteria Manual for Site Development and Construction that have been replaced by information in the TriSWM Appendix for use in the planning and design of stormwater management facilities for public transportation projects. Chapters or sections of the iSWM Criteria Manual for Site Development and Construction not referenced in the table are to be used "as is."

Affected Chapter/Section of the iSWM Criteria Manual	**Replacement Chapter/Section in TriSWM Appendix**	**Comments**
Chapter 1, Overview of iSWM Criteria Manual	Chapter 1, Overview of TriSWM Appendix	General content modifications as needed to reflect TriSWM requirements.
Chapter 2, *integrated* Development Process	Chapter 2, TriSWM Planning and Development Process	Complete section replaced; the project planning and development process for public facilities is significantly different than for private development projects.
Chapter 3, Section 3.2, Water Quality Protection	Chapter 3, Section 3.2, TriSWM Water Quality Protection	The Water Quality Protection Criteria has been modified due to the nature of linear facilities.
Chapter 3, Section 3.8, Stormwater Control Selection	Chapter 3, Section 3.8, TriSWM Stormwater Control Selection	The "*Ability to treat the Water Quality Volume"* section has been modified to reflect TriSWM water quality treatment designations. The tables have been changed as indicated below.
Table 3.6, Suitability of Stormwater Controls to Meet *integrated* Focus Areas, and Table 3.15, Stormwater Treatment Suitability[1]	Table 3.2, Stormwater Treatment Suitability[2]	Designations in the "Water Quality Protection" column have been changed to reflect TriSWM designations (Primary or Secondary changed to Levels I, II, or III). Also, *integrated* Stormwater Controls not typically associated with streets or roadways (Green Roofs, Rain Barrels, etc.) have been removed.

Affected Chapter/Section of the iSWM Criteria Manual	Replacement Chapter/Section in TriSWM Appendix	Comments
Table 3.16 Water Quality Performance	Table 3.3 Water Quality Performance	*integrated* Stormwater Controls not typically associated with streets or roadways have been removed.
Table 3.17 Site Applicability	Table 3.4 Site Applicability	
Table 3.18 Implementation Considerations	Table 3.5 Implementation Considerations	
Table 3.19 Physiographic Factors	Table 3.6 Physiographic Factors	
Table 3.20 Soils	Table 3.7 Soils	
Table 3.21 Special Watershed Considerations	Table 3.8 Special Watershed Considerations	
Table 3.22 Location and Permitting Checklist	Table 3.9 Location and Permitting Checklist	Minor updates for clarification.

1. Tables 3.6 and 3.15 in the iSWM Criteria Manual contain the same information and are both replaced by Table 3.2 in the TriSWM Appendix.
2. The Water Quality Protection designations for stormwater controls in Table 3.2 of the TriSWM Appendix shall also be used in place of the Water Quality Protection designations in Table 1.3 of the Stormwater Controls Technical Manual.

Note: Stormwater runoff from residential streets should be managed as part of the overall stormwater management system for the entire site. The iSWM Criteria Manual for Site Development and Construction should be used for the planning and design of stormwater management facilities for residential subdivisions and internal residential streets. The TriSWM Appendix does not apply to local or residential classified streets within residential subdivisions, unless required by the local jurisdiction. However, when a city or county cooperates with a developer in the construction of a collector or arterial street for access, the local government may require the use of the TriSWM Appendix for that portion of the project.

Local Provision Boxes

Throughout this manual there are “Local Provision” boxes. These boxes are used by a local government/agency to add, delete, or modify sections of the criteria and specify the options allowed and/or required by the local government/agency. Additional local information can be added at the back of this document.

Local Provisions:

Applicability

TriSWM is applicable under the following conditions for projects that will ultimately disturb one or more acres as indicated in Table 1.1.

Table 1.1 Applicability

Applicable for TriSWM Criteria :	Applicable for iSWM Construction Criteria:
Land disturbing activity of 1 acre or more **OR** land disturbing activity of less than 1 acre where the activity is part of a common plan of development that is one acre or larger.	Land disturbing activity of 1 acre or more **OR** land disturbing activity of less than 1 acre where the activity is part of a common plan of development that is one acre or larger. (Requirements located in Chapter 4, *integrated* Construction Criteria of the iSWM Criteria Manual for Site Development and Construction)

The criteria within the TriSWM Appendix is applicable to projects that disturb 1 acre or more, including projects less than one acre that are part of a larger common project plan or scope that will disturb 1 acre or more. A common plan of development consists of construction activity that is completed in separate stages, separate phases, or in combination with other construction activities.

Projects located in or near 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.

For some projects, particularly expansion projects, practical limitations may present obstacles to fully meeting stormwater management requirements within the project right-of-way (ROW). Limitations could include lack of land availability, engineering constraints, health and safety issues associated with operations and maintenance activities, or low benefit/cost ratio. If the project planning, assessment, and design process reveals that stormwater requirements for a project cannot be met because it is not feasible to do so, an explanation must be provided in the planning documents for the project. The explanation must include the reasons why the requirements cannot be met for the site and the provisions for stormwater management that can be provided.

Projects below Applicability Threshold

Projects that are below the size threshold for applicability requirements (above) are not subject to the water quality or streambank protection requirements of the TriSWM Appendix. However, it is recommended that these criteria still be used and that temporary controls be provided during construction. Flood mitigation and conveyance criteria still apply. The planning process is also simplified for sites below the applicable criteria to an optional pre-development review before the final submittal of the engineering plans.

Local Provisions:

1.2 TriSWM Development Process

Chapter 2 presents information on the process of collecting and considering appropriate information needed to effectively and efficiently manage stormwater on roadway, street, and highway projects. Descriptions of the city/county and Texas Department of Transportation (TxDOT) project development processes are provided along with information on site analysis and inventory, conditions for accepting off-site flows, and special planning and design considerations.

Local Provisions:

1.3 TriSWM Design Criteria

Chapter 3 presents an approach for meeting stormwater runoff quality and quantity management goals by addressing the key adverse impacts of development on stormwater runoff. Its framework consists of three focus areas, each with options in terms of how the focus area is applied.

Design Focus Areas

The stormwater management focus areas and goals are:

- **Water Quality Protection:** Remove or reduce pollutants in stormwater runoff to protect water quality
- **Streambank Protection:** Regulate discharge from the site to minimize downstream bank and channel erosion
- **Flood Mitigation and Conveyance:** Control runoff within and from the site to minimize flood risk to people and properties for the conveyance storm as well as the 100-year storm.

Each of the Design Focus Areas must be used in conjunction with the others to address the overall stormwater impacts from a development site. When used as a set, the Design Focus Areas control the entire range of hydrologic events, from the smallest runoff-producing rainfalls up to the 100-year, 24-hour storm.

Local Provisions:

Design Storms

TriSWM design is based on the following four (4) storm events.

Table 1.2 Storm Events	
Storm Event Name	**Storm Event Description**
"Water Quality"	Criteria based on a volume of 1.5 inches of rainfall, not a storm frequency
"Streambank Protection"	1-year, 24-hour storm event
"Conveyance"	25-year, 24-hour storm event
"Flood Mitigation"	100-year, 24-hour storm event

Throughout the manual the storms will be referred to by their storm event names.

Local Provisions:

Design Focus Area Application Options

There are multiple options provided to meet the required criteria for water quality protection, streambank protection, and flood mitigation. Design requirements and options are summarized in Table 1.3.

Design criteria for streambank protection and flood mitigation are based on a **downstream assessment**. The purpose of the downstream assessment is to protect downstream properties and channels from increased flooding and erosion potential due to the proposed project. A downstream assessment is required to determine the extent of improvements necessary for streambank protection and flood mitigation. Downstream assessments shall be performed for streambank protection, conveyance, and flood mitigation storm events. More information on downstream assessments is provided in Section 3.3. of the iSWM Criteria Manual for Site Development and Construction

If a project causes no adverse impacts to existing conditions, then it is possible that little or no mitigation would be required.

Table 1.3 Summary of Options for Design Focus Areas

Design Focus Area	Reference Section	Required Downstream Assessment	Design Requirements/Options
Water Quality Protection	**3.2** TriSWM Appendix	no	Water Quality Protection requirements are determined based on the quality of receiving waters, proximity of project discharge to any wetlands and/or drinking water supply intakes, and projected traffic volume. Refer to Section 3.2 to determine the Water Quality Treatment Level required (Treatment Level I, II, or III).
Streambank Protection	**3.4** iSWM Criteria Manual	yes	**Option 1:** Reinforce/stabilize downstream conditions
			Option 2: Install stormwater controls to maintain or improve existing downstream conditions
			Option 3: Provide on-site controlled release of the 1-year, 24-hour storm event over a period of 24 hours (Streambank Protection Volume, SP_V)
Flood Mitigation and Conveyance	**3.5 and 3.6** iSWM Criteria Manual	yes	**Flood Mitigation** **Option 1:** Provide adequate downstream conveyance systems
			Option 2: Install stormwater controls on-site to maintain or improve existing downstream conditions
			Option 3: In lieu of a downstream assessment, maintain existing on-site runoff conditions
			Conveyance Minimize localized site flooding of streets, sidewalks, and properties by a combination of on-site stormwater controls and conveyance systems

Local Provisions:

2.0 TriSWM Development Process

2.1 Project Development Goals

In order to most effectively and efficiently manage stormwater on new public roadway, street, and highway projects, as well as significant expansion projects, consideration of stormwater runoff needs to be fully integrated into the project planning and design process. This involves a comprehensive planning approach and a thorough understanding of the physical characteristics and natural resources in proximity to the proposed route. In addition, the management of the quantity and the quality of stormwater should be addressed in an integrated approach. The purpose of the TriSWM Appendix is to provide design guidance and a framework for incorporating effective and environmentally sensitive stormwater management into the street and highway project development process and to encourage a greater uniformity in developing plans for stormwater management systems that meet the following goals:

- Provide safe driving conditions
- Minimize the downstream flood risk to people and properties
- Minimize downstream bank and channel erosion
- Reduce pollutants in stormwater runoff to protect water quality.

2.2 Stormwater Management Planning

2.2.1 Introduction

The planning phase offers the greatest opportunity to avoid adverse water quality impacts as alignments and right-of-way requirements are developed and refined. Conducting natural and cultural resource studies concurrently with early project planning provides timely information to assist in identifying and avoiding potential impacts. Sections 2.2.6, Site Analysis and Inventory, and 2.3, Special Planning and Design Considerations, describe the features that should be considered and avoided if possible. Avoiding impacts may reduce or eliminate the need for higher level water quality treatment controls.

Once the alignment has been determined, planning and design of stormwater management controls should be performed early in the preliminary design phase of the project so that adequate right-of-way may be acquired. This would generally be at the site assessment and preliminary design phases of a city/county street project or the preliminary design phase of a TxDOT project. The proposed alignment should include sufficient reserved land to construct and maintain all required BMPs at appropriate locations.

Local Provisions:

2.2.2 City / County Project Development Process

Local governments plan for the preservation and creation of transportation corridors through master thoroughfare plans and/or comprehensive plans. The function of these planning tools is to establish the future roadway network and design guidelines to provide an adequate level of service. Thoroughfare planning is used by local government to proactively prepare for future traffic conditions, accommodate growth and development and identify projects for the capital improvements program (CIP), determine roadway right-of-way requirements, and improve community aesthetics and safety. Conventional thoroughfare planning should be expanded to include avoidance of sensitive natural features where possible and to accommodate stormwater management best management practices (BMPs).

Planning for individual projects typically starts with identification in the capital improvement program, which is a long-range financial planning tool to address community needs in the long-term future for improving streets, drainage, parks, public facilities, utilities and other city functions. Projects selected for funding in the CIP would proceed through various stages of development including Site Assessment, Preliminary Design, Right-of-Way Acquisition, Final Design, and Drawings & Specifications.

The Site Assessment phase consists of identifying physical and environmental constraints on the potential alignment of the project. The Preliminary Design phase incorporates information from the site assessment and identifies the vertical alignment for the street or roadway. Typically, preliminary design drawings are reviewed by the local government at a point where the engineering design is approximately 30 to 50 percent complete. Once the preliminary plans and vertical alignment are approved, activities to acquire the right-of-way are initiated. While right-of-way acquisition efforts are in progress, the final design drawings and specifications for the project are completed and reviewed by the local government.

Since many stormwater management best management practices require additional space beyond the typical right-of-way (50' two-lane streets, 120 – 130' for 6-lane divided with median), stormwater management practices must be identified during the Preliminary Design phase. Once stormwater management controls are identified, the right-of-way acquisition process and development of the final design may proceed accordingly.

Local Provisions:

2.2.3 TxDOT Project Development Process

The TxDOT project development process is laid out in detail in the Project Development Process Manual, which may be accessed at http://onlinemanuals.txdot.gov/txdotmanuals/pdp/index.htm. A general characterization of the process is outlined below:

- Planning and Programming
  Consists of needs identification, site visit, project authorization, compliance with planning requirements, determination of study requirements, and construction funding identification.
- Preliminary Design
  Consists of data collection and preliminary design preparation, public meetings, preliminary schematic preparation, geometric schematic preparation (including determination of right-of-way needs), and value engineering. Development of the preliminary and geometric schematics is a particularly important phase since alternative alignments are evaluated, ROW and access control requirements are defined, and initial siting and sizing of permanent stormwater BMPs must be determined.
- Environmental
  Consists of environmental issues determination and data collection, interagency coordination and permitting, environmental documentation, public hearing, and environmental clearance. This process is further described below.
- Right-of-Way and Utilities
  Consists of right-of-way and utility data collection, mapping, appraisals and acquisition, and utility adjustments.

- Plans, Specifications, and Engineering Development
  Consists of the design conference, design of bridges, final vertical and horizontal alignment design, roadway design, drainage design, and final review.
- Letting
  Consists of final funding approval and bidding and award of construction contract.

The project development process is overseen by the District's Area Engineer and Project Manager. The District Environmental Quality Coordinator (DEQC) reviews project plans prior to letting to ensure that the Stormwater Pollution Prevention Plan and Environmental Permits, Issues, and Commitments (EPIC) plan sheets are complete. The EPIC sheet is used to summarize the special requirements and restrictions related to the construction activity that has been permitted and the conditions of any permits. For example, it may depict areas to be avoided during construction due to the presence of endangered species, wetlands, etc. The DEQC and divisional and central management are aided by the Environmental Compliance Oversight System (ECOS). It's a database system that tracks the environmental process for projects generated by TxDOT's 25 Districts. The ECOS tracks and facilitates coordination throughout the TxDOT system concerning:

- Project environmental clearance
- Environmental Permits, Issues and Commitments (EPIC)
- Public involvement
- Cultural resources protection
- Hazardous material avoidance or removal
- Corps of Engineers permits
- Biological resource protection
- Water quality protection
- Coordination with other regulatory agencies as necessary

Local Provisions:

2.2.4 Determine/Confirm Local Requirements

The consultant or project designer must determine the stormwater management requirements of the jurisdiction(s) that the project will be located in. For local governments that have adopted the iSWM™ Criteria Manual for Site Development and Construction, much of this information is available in the jurisdiction's adopted version of the iSWM Criteria Manual. These requirements may include:

- Design storm frequencies
- Conveyance design criteria
- Floodplain criteria
- Buffer/setback criteria
- Watershed-based criteria
- Need for physical site evaluations such as infiltration tests, geotechnical evaluations, etc.

Local Provisions:

2.2.5 Conditions for Accepting Off-Site Flows

Local governments and the Texas Department of Transportation (TxDOT) must provide for the passage of off-site flows through street and highway right-of-way to maintain natural drainage paths. If a private developer's project discharges off-site flow to public right-of-way, local governments designated as Municipal Separate Storm Sewer Systems (MS4s) must require the private development project to comply with the requirements of the *integrated* Stormwater Management (iSWM™) Criteria Manual for Site Development and Construction (if adopted) or other local government post construction stormwater quality management requirements. Once the local government MS4 accepts discharge of water onto its right-of-way, the jurisdiction becomes liable for the quality of that discharge under Texas Pollutant Discharge Elimination System (TPDES) regulations.

TxDOT lacks statutory authority to prohibit or control post-construction discharges of stormwater from development projects outside the right-of-way. TxDOT should coordinate with local governments to the extent possible to ensure that private development projects meet the jurisdiction's post construction stormwater management requirements.

Local Provisions:

2.2.6 Site Analysis and Inventory

Using approved field and mapping techniques, the project designer 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 / receiving waters
- Stream flow data
- Soils
- Ground cover and vegetation
- Wetlands
- Critical habitat areas
- Boundaries of wooded areas
- Floodplain boundaries
- Steep slopes
- Required buffers

- Other required protection areas (e.g., well setbacks)
- Clean Water Act Section 303(d) listed impaired stream segments
- Proposed stream crossing locations
- Existing stormwater facilities (open channels & enclosed)
- Existing development
- Utilities
- Adjacent areas
- Property lines and easements

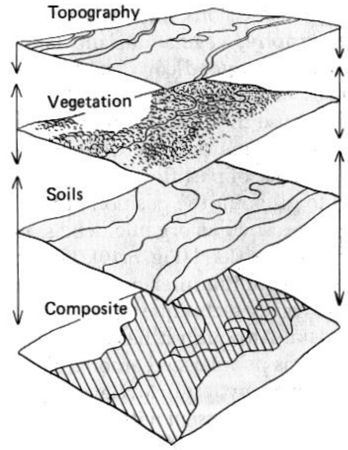


Figure 1.1
Composite Analysis
(Source: Marsh, 1983)

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 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 once (see Figure 1.1).

Local Provisions:

2.3 Special Planning and Design Considerations

This section discusses several environmental features that need to be identified and assessed during the earliest stages of planning for a project, as well as design considerations for bridges and right-of-way. Proposed alignments for a project should avoid sensitive natural resources to the greatest extent practicable. In cases where avoidance is not possible, providing an undisturbed buffer and additional practices or structural controls to minimize impact must be considered.

Preserving natural conservation areas such as undisturbed forested and vegetated areas, floodplains, stream corridors and wetlands helps to preserve the original hydrology and avoids the impact of stormwater runoff and pollutants. Undisturbed vegetated areas also stabilize soils, provide for filtering and infiltration, decreases evaporation, and increases transpiration.

Buffer areas and sensitive features in proximity to project alignments should be 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.

Projects located in or near 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.

For some projects, particularly expansion projects, practical limitations may present obstacles to fully meeting stormwater management requirements within the project right-of-way (ROW). Limitations could include lack of land availability, engineering constraints, health and safety issues associated with operations and maintenance activities, or low benefit/cost ratio. If the project planning, assessment, and design process reveals that stormwater requirements for a project cannot be met because it is not feasible to do so, an explanation must be provided in the planning documents for the project. The explanation must include the reasons why the requirements cannot be met for the site and the provisions for stormwater management that can be provided.

Local Provisions:

2.3.1 Sensitive Areas

Stream segments classified by the Texas Commission on Environmental Quality (TCEQ) as Exceptionally-High quality should be avoided if possible when considering potential alignments. These are waters that have been designated "Exceptional Quality Aquatic Habitat" by the TCEQ or "Endangered/Protected Species Habitat" by the Texas Parks and Wildlife Department.

- Exceptional Quality Aquatic Habitat – segments that are significant due to unique or critical habitats and exceptional aquatic life uses dependent on or associated with high water quality
- Endangered/Protected Species Habitat – sites along segments where water development projects would have significant detrimental effects on state or federally listed threatened and endangered species, and sites along segments that are significant due to the presence of unique, exemplary, or unusually extensive natural communities

Local Provisions:

2.3.2 Wetlands

Because the alteration of ground cover and drainage patterns will almost always affect the hydrology of wetlands, and because hydrologic changes strongly impact vegetation and amphibian communities, it is always preferable to avoid wetland areas when determining road or street alignments if possible.

An important measure to maintain the health of a natural wetland is the protection and control of the wetland's hydroperiod. The hydroperiod is the pattern of fluctuation of water depth and the frequency and duration of drying in the summer. A hydrological assessment is performed to determine pre-project hydroperiod characteristics and to model the post-project conditions. Coordination with the TCEQ is necessary to properly assess the impact of hydroperiod changes.

The design of facilities adjacent to wetlands should maximize natural water storage and infiltration opportunities within the project area. Natural wetlands may not be used in lieu of runoff treatment BMPs. Any construction of stormwater treatment or flow control facilities is discouraged within natural wetland areas, with the exception of the following situations, which involve additional permitting:

- Necessary conveyance systems with applicable permits
- Lower quality wetland approved for hydrologic modification

Local Provisions:

2.3.3 Floodplains

Development in floodplain areas can reduce the ability of the floodplain to convey stormwater, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. 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.

Roadway construction can displace hydrologic storage, resulting in increased stream flows, erosion, and decreased infiltration. Loss of hydrologic storage may require creation of additional hydrologic storage elsewhere in the watershed. Design for management of stormwater runoff from transportation facilities in floodplains differs from parcel based BMPs primarily in the increased influence of off-site stormwater entering the facility, space limitations of a linear facility, and the likelihood that roadways will cross jurisdictional boundaries.

Local Provisions:

2.3.4 Aquifers and Wellhead Protection Areas

Pollutants can enter aquifers through stormwater runoff treatment and storage systems. Local ordinances may specify minimum setbacks or buffers between wellheads and roadway construction. In Texas, the TCEQ's Source Water Assessment Program (SWAP), Source Water Protection Program (SWP) and Wellhead Protection Program (WHP) may also impact BMP selection and implementation for transportation projects. Aquifer recharge zones may also have state or local restrictions.

Local Provisions:

2.3.5 Streams and Riparian Areas

Roadway alignments should cross streams and riparian areas as few times as possible and should be located a sufficient distance from the stream when the alignment is parallel. Maintaining riparian buffers is important for the protection of stream banks and stream ecosystems.

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 stormwater to concentrate and flow into the stream without first flowing through the 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. Any structural controls for management of stormwater should be located outside the riparian buffer if possible.

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.

Local Provisions:

2.3.6 Impaired Water Bodies

Impaired water bodies are those surface waters identified in the *Texas Integrated Report of Surface Water Quality for Clean Water Act Sections 305(b) and 303(d)* as not meeting water quality standards. In compliance with the federal Clean Water Act, the Texas Commission on Environmental Quality (TCEQ) researches, updates, and then publishes the list every two years. Impaired water bodies are eventually assigned a Total Maximum Daily Load (TMDL), which is the maximum amount of the impairing pollutant that the water body can receive and still comply with water quality standards. There are several impaired water bodies in the Dallas-Fort Worth metropolitan area, including those with and without TMDLs. Impairments may be for a variety of pollutants including bacteria and legacy pollutants such as PCBs and dioxin. Discharges of stormwater runoff containing pollutants of concern (any pollutant identified as a cause of impairment) to impaired water bodies will be governed by an entity's Texas Pollutant Discharge Elimination System (TPDES) Municipal Separate Storm Sewer System (MS4) permit, if applicable.

Local Provisions:

2.3.7 Facilities Designated as Hazardous Materials Routes

Shipments of hazardous materials along roadways that are listed on the National Hazardous Material Route Registry have the potential for accidental release of hazardous materials. Hazardous material traps should be considered for placement depending on the level of sensitivity of receiving waters, the probability of spills, and the nature of the stormwater collection system (particularly if the road surface drains directly to inlet and pipe system that discharge to surface waters). Gravity or other proprietary oil-water separators provide some level of protection, but the capacity may be exceeded and these devices are also generally not effective at containing corrosives. For maximum protection of sensitive areas, detention basins lined with clay, concrete, or other impermeable liner with a capture volume of at least 10,000 gallons should be considered.

Local Provisions:

2.3.8 Bridges

The portion of bridge stormwater runoff associated with the part of the bridge over water is the same volume as would have fallen in the water body without the presence of the bridge. The water quality, however, is impacted by material deposited on the road surface. Furthermore, the bridge itself doesn't offer an opportunity for treatment or infiltration. Although bridges have traditionally been built with gutters routing stormwater directly into the receiving waters, this is no longer the preferred alternative. It is recommended that runoff be collected and conveyed to the ends of the bridge and directed to the selected treatment facility as necessary. Collection and conveyance systems must be designed to prevent backup of stormwater onto the bridge surface in the event of clogging by trash and debris.

Local Provisions:

2.3.9 Right-of-Way

After the stormwater treatment requirements of the project are determined, and the hydrology of the site is known, the area required for stormwater treatment facilities can be estimated. Availability and cost of right-of-way may influence treatment selection. Placement of the roadway and stormwater treatment facilities within the right-of-way can be adjusted and additional right-of-way requirements may be identified.

Local Provisions:

2.3.10 Protection of Permanent Stormwater Controls during Construction

Permanent stormwater controls must be protected from damage due to excess sedimentation during construction of the project. All disturbed areas upstream of permanent stormwater controls should ideally achieve final stabilization prior to stormwater runoff being permitted to flow into the permanent control. At a minimum, permanent stormwater controls receiving runoff from disturbed areas must be protected by sediment controls such as silt fence or filter tubes. Permanent stormwater controls must be fully operational (no sediment buildup, no clogged filter media, plant material in place, proper infiltration rates achieved, etc.) as a condition of project acceptance from the contractor.

Local Provisions:

3.0 TriSWM Design Criteria

3.1 Hydrologic Methods

Refer to the iSWM Criteria Manual for Site Development and Construction, Section 3.1, Hydrologic Methods.

3.2 TriSWM Water Quality Protection

3.2.1 Water Quality Treatment Level Criteria

In assessing the need to incorporate post-construction water quality control measures into street and highway construction projects, the quality of receiving waters is to be considered along with projected traffic volume for the facility. Of many variables that affect the quality of runoff from a roadway (rainfall characteristics, traffic type, surrounding land use, etc.), average daily traffic volume (ADT) is a determining factor for which data is readily available.

Various studies and reports published by the Federal Highway Administration have concluded that greater pollutant levels in stormwater runoff could be anticipated where traffic volume exceeds 30,000 ADT. Therefore, 30,000 vehicles per day (VPD) is used as the threshold between low volume and high volume roadways and the corresponding level of post-construction stormwater quality treatment required.

The water quality of streams or reservoirs and existence of downstream critical areas are used to classify receiving waters and riparian environments. The classification is based on the susceptibility of the receiving waters and riparian areas to negative impact from pollutants in stormwater runoff from the proposed project. The classification of receiving waters is as follows:

1. **High**: These are receiving waters that meet one or more of the following criteria:
    - Designated as “Exceptional Quality Aquatic Habitat” by the TCEQ
    - Identified as Endangered/Protected Species Habitat by the Texas Parks and Wildlife Department
    - Proximity and potential impact to drinking water supply reservoir (as determined by water treatment provider)

2. **Moderate**: These are receiving waters that meet one or more of the following criteria:
    - Three or more designated uses on the Texas Surface Water Quality Standards, or any perennial stream* not classified on the Texas Surface Water Quality Standards
    - Wetlands located on the project site or downstream of the project where flow from the project would constitute more than 10% of total flow to the wetland

3. **Minimal**: All receiving waters not categorized above, including receiving waters listed with two or less designated uses on the Texas Surface Water Quality Standards and intermittent streams*

* Intermittent stream: A stream that has a period of zero flow for at least one week during most years. Perennial stream: A stream that has flow nearly continually (does not reach zero flow for one week or more) during most years.

Table 3.1 shows the level of post-construction stormwater management measures required for street and highway projects based on the previously discussed factors of traffic volume and quality of receiving waters. The levels should be considered during project planning and design for construction of new streets and highways and major reconstruction projects. The ADT will be based on a 20-year design projection.

Table 3.1 Post-Construction Water Quality Treatment Levels			
Traffic Volume	**Receiving Water / Riparian Area Susceptibility**		
	Minimal	**Moderate**	**High**
Low (<30,000 VPD)	Level I	Level I	Level II
High (>30,000 VPD)	Level I	Level II	Level III

Once the treatment level requirements have been established for the project, select practices or structural stormwater controls in accordance with the appropriate category. Section 3.8 and the *Site Development Controls Technical Manual* contain selection, pollutant removal effectiveness, and design information for the structural controls listed.

Treatment Level I

Select one or more of the following practices and/or structural controls:

- Program of Scheduled Pollution Prevention Practices
  Municipal pollution prevention/good housekeeping practices such as street sweeping, storm drain inlet cleaning, and proper application of landscape chemicals
- Off-site Pollution Prevention Activities/Programs
  Route stormwater runoff to new or existing watershed-level BMPs (i.e. regional detention, Dallas CBD sumps, etc.) identified in the entity's MS4 Permit / Stormwater Management Program
- Grass Channels
- Filter Strips
- Gravity (Oil-Grit) Separator
- Proprietary Structural Controls
- Porous Concrete / Modular Porous Paver Systems

Treatment Level II

Select one or more of the following practices and/or structural controls:

- Enhanced Swales
- Bioretention Areas
- Dry Detention / Extended Detention Dry Basins
- Supplement with any BMPs identified in Level I

Treatment Level III

Select one or more of the following practices and/or structural controls:

- Organic Filter
- Sand Filter
- Underground Sand Filter
- Infiltration Trenches
- Stormwater (Wet) Ponds
- Stormwater Wetlands
- Alum Treatment Systems (used as pretreatment in conjunction with wet pond)
- Supplement with any BMPs identified in Levels I and II

Once the treatment level is established and potential practices and structural controls are identified, the volume of runoff to be treated must be calculated in accordance with the following section for some controls. Refer to the *Site Development Controls Technical Manual* for each of the proposed controls to determine whether the water quality protection volume is applicable. Structural controls or practices from a higher Treatment Level category may be used to meet lower Treatment Level requirements if desired. Combinations of practices and controls may also be implemented. A detailed discussion of each of the controls, as well as design criteria and procedures, can be found in the *Site Development Controls Technical Manual*.

Local Provisions:

3.2.2 Water Quality Protection Volume

Treat the Water Quality Protection Volume by reducing total suspended solids from the development site for runoff resulting from rainfall of 1.5 inches (85th percentile storm). Stormwater runoff equal to the Water Quality Protection Volume generated from sites must 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.

The Water Quality Protection Volume (WQ_v) is the runoff from the first 1.5 inches of rainfall. Thus, a stormwater 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. For methods to determine the WQ_v, see *Section 1.2 of the Water Quality Technical Manual*.

Local Provisions:

3.2.3 Stormwater Controls Overview

This section provides an overview of stormwater controls used to address stormwater quality, as well as streambank protection and flood mitigation, which are covered in Sections 3.4 and 3.5 of the iSWM Criteria Manual for Site Development and Construction. Table 3.2, Stormwater Treatment Suitability (located in Section 3.8.1 of the TriSWM Appendix) summarizes the stormwater management suitability of the various stormwater controls in addressing the stormwater Focus Areas. The *Site Development Controls Technical Manual* provides guidance on the use of stormwater controls as well as how to calculate the pollutant removal efficiency for stormwater controls in series. The *Site Development Controls Technical Manual* also provides guidance for choosing the appropriate stormwater control(s) for a site as well as the basic considerations and limitations on the use of a particular stormwater control.

The stormwater control practices recommended in this manual vary in their applicability and ability to meet stormwater management goals:

Water Quality Protection
Stormwater Controls are classified as Level I, Level II, or Level III depending on the ability of the control to achieve the desired reduction in pollutants. When designed to treat the required Water Quality Volume

(WQ_v) and constructed and maintained in accordance with recommended specifications, the desired level of protection is presumed to be provided to the receiving waters.

Streambank Protection and Flood Control
Stormwater Controls designated as “Primary” controls have the ability to fully address one or more of the Steps in the TriSWM Planning and Design Approach if designed appropriately. Several of these structural controls can be designed to provide primary control for downstream streambank protection (SPv) and flood control (Qf). These structural controls are recommended stormwater management facilities for a site wherever feasible and practical.

Stormwater Controls designated as “Secondary” controls are recommended only 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 a specifc stormwater Focus Area, (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.

Using Other or New Structural Stormwater Controls
Local governments and agencies can utilize controls not included in this guide at their discretion. Such controls may be utilized if independent performance data shows that the structural control conforms to requirements for treatment, conveyance, maintenance, and environmental impact.

Local Provisions:

3.3 Acceptable Downstream Conditions

Refer to the iSWM Criteria Manual for Site Development and Construction, Section 3.3, Acceptable Downstream Conditions.

3.4 Streambank Protection

Refer to the iSWM Criteria Manual for Site Development and Construction, Section 3.4, Streambank Protection.

3.5 Flood Mitigation

Refer to the iSWM Criteria Manual for Site Development and Construction, Section 3.5, Flood Mitigation.

3.6 Stormwater Conveyance Systems

Refer to the iSWM Criteria Manual for Site Development and Construction, Section 3.6, Stormwater Conveyance Systems.

3.7 Easements, Plats, and Maintenance Agreements

Refer to the iSWM Criteria Manual for Site Development and Construction, Section 3.7, Easements, Plats, and Maintenance Agreements.

3.8 TriSWM Stormwater Control Selection

3.8.1 Control Screening Process

Outlined below is a screening process for structural stormwater controls that 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 to provide guidance on factors to consider in their location. This information is also contained in the *Site Development Controls Technical Manual*.

The following four criteria shall be evaluated in order to select the appropriate structural control(s) or group of controls for a development:

- Stormwater treatment suitability
- Water quality performance
- Site applicability
- Implementation considerations

In addition, the following factors shall be considered for a given site and any specific design criteria or restrictions need to be evaluated:

- Physiographic factors
- Soils
- Special watershed or stream considerations

Finally, environmental regulations shall be considered as they may influence the location of a structural control on site or may require a permit.

The following steps provide a selection process for comparing and evaluating various structural stormwater 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 stormwater management and design objectives for a development site or project.

Step 1 Overall Applicability

The following are the details of the various screening categories and individual characteristics used to evaluate the structural controls.

Table 3.2 – Stormwater Treatment Suitability

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* Focus Area. This does not necessarily mean that it should be eliminated from consideration, but rather it 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 provide water quality protection: Stormwater Controls are classified as Level I, Level II, or Level III depending on the ability of the control to achieve the desired reduction in pollutants. When designed to treat the required Water Quality Volume (WQ_v) and constructed and maintained in accordance with recommended specifications, the desired level of protection is presumed to be provided to the receiving waters.

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 flood mitigation storm event.

Table 3.3 - Relative Water Quality Performance

The second category of the matrix provides an overview of the pollutant removal performance for 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 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 concern when meeting regulatory water 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 that 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 in the *Site Development Controls Technical Manual*. Local jurisdictions may have other site uses that they designate as hotspots. Therefore, their criteria should be checked as well.

Table 3.4 - 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.

Table 3.5 - 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 stormwater 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.

Local Provisions:

Table 3.2 Stormwater Treatment Suitability						
Category	**Stormwater Controls**	**TSS/ Sediment Removal Rate**	**Water Quality Protection#**	**Streambank Protection**	**On-Site Flood Control**	**Downstream Flood Control**
Bioretention Areas	Bioretention Areas	80%	Level II	S	S	-
Channels	Enhanced Swales	80%	Level II	S	S	S
	Channels, Grass	50%	Level I	S	P	S
	Channels, Open	-	-	-	P	S
Chemical Treatment	Alum Treatment System	90%	Level III	-	-	-
Conveyance System Components	Culverts	-	-	-	P	P
	Energy Dissipation	-	-	P	S	S
	Inlets/Street Gutters	-	-	-	P	-
	Pipe Systems	-	-	P	P	P
Detention	Detention, Dry	65%	Level II	P	P	P
	Detention, Extended Dry	65%	Level II	P	P	P
	Detention, Multi-purpose Areas	-	-	P	P	P
	Detention, Underground	-	-	P	P	P
Filtration	Filter Strips	50%	Level I	-	-	-
	Organic Filters	80%	Level III	-	-	-
	Sand Filters, Surface/Perimeter	80%	Level III	S	-	-
	Sand Filters, Underground	80%	Level III	-	-	-
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	40%	Level I	-	-	-
Infiltration	Infiltration Trenches	80%	Level III	S	-	-
Ponds	Wet Pond	80%	Level III	P	P	P
	Wet ED Pond	80%	Level III	P	P	P
	Micropool ED Pond	80%	Level III	P	P	P
Porous Surfaces	Modular Porous Paver Systems	[2]	Level I	S	-	-
	Porous Concrete	[2]	Level I	S	-	-
Proprietary Systems	Proprietary Systems [1]	[1]	Level I	S	S	S
Wetlands	Wetlands, Stormwater	80%	Level III	P	P	P
	Wetlands, Submerged Gravel	80%	Level III	P	S	-

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

S = Secondary Control: May partially meet design criteria. Designated as a Secondary control due to considerations such as maintenance concerns. For Water Quality Protection, recommended for limited use in approved community-designated areas.

\# = Applicability of controls to meet Water Quality Treatment Level Criteria.

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

[1] = 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, if used as a primary control. Third-party sources could include Technology Acceptance Reciprocity Partnership, Technology Assessment Protocol – Ecology, or others.

[2] = Porous surfaces provide water quality benefits by reducing the effective impervious area.

Table 3.3 Water Quality Performance

Category	Stormwater Controls	Water Quality Performance			
		TSS/ Sediment Removal Rate	Nutrient Removal Rate (TP/TN)	Bacteria Removal Rate	Hotspot Application
Bioretention Areas	Bioretention Areas	80%	60%/50%	-	✓
Channels	Enhanced Swales	80%	25%/40%	-	✓
	Channels, Grass	50%	25%/20%	-	
	Channels, Open	-	-	-	
Chemical Treatment	Alum Treatment System	90%	80%/60%	90%	✓
Conveyance System Components	Culverts	-	-	-	
	Energy Dissipation	-	-	-	
	Inlets/Street Gutters	-	-	-	
	Pipe Systems	-	-	-	
Detention	Detention, Dry	65%	50%/30%	70%	✓
	Detention, Extended Dry	65%	50%/30%	70%	✓
	Detention, Multi-purpose Areas	-	-	-	
	Detention, Underground	-	-	-	
Filtration	Filter Strips	50%	20%/20%	-	
	Organic Filters	80%	60%/40%	50%	✓
	Sand Filters, Surface/Perimeter	80%	50%/25%	40%	✓
	Sand Filters, Underground	80%	50%/25%	40%	✓
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	40%	5%/5%	-	
Infiltration	Infiltration Trenches	80%	60%/60%	90%	
Ponds	Wet Pond	80%	50%/30%	70%	✓
	Wet ED Pond	80%	50%/30%	70%	✓
	Micropool ED Pond	80%	50%/30%	70%	✓
Porous Surfaces	Modular Porous Paver Systems	[2]	80%/80%	-	
	Porous Concrete	[2]	50%/65%	-	
Proprietary Systems	Proprietary Systems [1]	[1]	[1]	[1]	
Wetlands	Wetlands, Stormwater	80%	40%/30%	70%	✓
	Wetlands, Submerged Gravel	80%	40%/30%	70%	✓

✓ = Meets suitability criteria

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

[1] = 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 if used as a primary control.

[2] = Porous surfaces provide water quality benefits by reducing the effective impervious area.

Table 3.4 Site Applicability

Category	Stormwater Controls	Site Applicability				
		Drainage Area (acres)	Space Req'd (% of Tributary imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table
Bioretention Areas	Bioretention Areas	5 max[3]	5-7%	6% max	5 ft	2 ft
Channels	Enhanced Swales	5 max	10-20%	4% max	1 ft	Below WT
	Channels, Grass					
	Channels, Open					
Chemical Treatment	Alum Treatment System	25 min	None			
Conveyance System Components	Culverts					
	Energy Dissipation					
	Inlets/Street Gutters					
	Pipe Systems					
Detention	Detention, Dry		2-3%	15% across pond	6 to 8 ft	2 ft
	Detention, Extended Dry		2-3%	15% across pond	6 to 8 ft	2 ft
	Detention, Multi-purpose Areas	200 max		1% for Parking Lot; 0.25 in/ft for Rooftop		
	Detention, Underground	200 max				
Filtration	Filter Strips	2 max[3]	20-25%	2-6%		
	Organic Filters	10 max[3]	2-3%		5 to 8 ft	
	Sand Filters, Surface/Perimeter	10 max[3] / 2 max[3]	2-3%	6% max	5 ft per 2-3 ft	2 ft
	Sand Filters, Underground	5 max	None			
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	1 max[3]	None			
Infiltration	Infiltration Trenches	5 max	2-3%	6% max	1 ft	4 ft
Ponds	Wet Pond		2-3%	15% max	6 t 8 ft	2 ft, if hotspot or aquifer
	Wet ED Pond	25 min[3]				
	Micropool ED Pond	10 min[3]				
Porous Surfaces	Modular Porous Paver Systems	5 max	Varies			
	Porous Concrete	5 max	Varies			
Proprietary Systems	Proprietary Systems [1]	[1]	[1]			
Wetlands	Wetlands, Stormwater	25 min	3-5%	8% max	3 to 5 ft (shallow) 6 to 8 ft (pond)	2 ft, if hotspot or aquifer
	Wetlands, Submerged Gravel	5 min			2 to 3 ft	Below WT

- = Not typically used or able to meet design criterion.
[1] = 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 if used as a primary control.
[2] = Porous surfaces provide water quality benefits by reducing the effective impervious area.
[3] = Drainage area can be larger in some instances

Table 3.5 Implementation Considerations

Category	Stormwater Controls	Implementation Considerations			
		Residential Subdivision Use	High Density/Ultra Urban	Capital Cost	Maintenance Burden
Bioretention Areas	Bioretention Areas	✓	✓	Moderate	Low
Channels	Enhanced Swales	✓		High	Low
	Channels, Grass	✓		Low	Moderate
	Channels, Open	✓		Low	Low
Chemical Treatment	Alum Treatment System	✓	✓	High	High
Conveyance System Components	Culverts	✓	✓	Low	Low
	Energy Dissipation	✓	✓	Low	Low
	Inlets/Street Gutters	✓	✓	Low	Low
	Pipe Systems	✓	✓	Low	Low
Detention	Detention, Dry	✓		Low	Moderate to High
	Detention, Extended Dry	✓		Low	Moderate to High
	Detention, Multi-purpose Areas	✓	✓	Low	Low
	Detention, Underground		✓	High	Moderate
Filtration	Filter Strips	✓		Low	Moderate
	Organic Filters		✓	High	High
	Sand Filters, Surface/Perimeter		✓	High	High
	Sand Filters, Underground		✓	High	High
Hydrodynamic Devices	Gravity (Oil-Grit) Separator		✓	High	High
Infiltration	Downspout Drywell	✓	✓	Low	Moderate
	Infiltration Trenches	✓	✓	High	High
	Soakage Trenches	✓	✓	High	High
Ponds	Wet Pond	✓		Low	Low
	Wet ED Pond	✓		Low	Low
	Micropool ED Pond	✓		Low	Moderate
	Multiple Ponds	✓		Low	Low
Porous Surfaces	Green Roof		✓	High	High
	Modular Porous Paver Systems		✓	Moderate	High
	Porous Concrete		✓	High	High
Proprietary Systems	Proprietary Systems [1]	[1]	✓	High	High
Re-Use	Rain Barrels	✓	✓	Low	High
Wetlands	Wetlands, Stormwater	✓		Moderate	Moderate
	Wetlands, Submerged Gravel	✓	✓	Moderate	High

✓ = Meets suitability criteria
\- = Not typically used or able to meet design criterion.
[1] = 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 if used as a primary control.

Step 2 Specific Criteria

The last three categories in the Stormwater Control Screening matrix provide 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.

Table 3.6 - 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 Somervell 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 stormwater 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.

Table 3.7 - 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.

Table 3.8 - Special Watershed or Stream Considerations

The design of stormwater controls is fundamentally influenced by the nature of the downstream water body that will be receiving the stormwater 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 stormwater 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 available treatment, a greater level of pollutant removal may be necessary for the pollutants of concern, such as bacteria pathogens, nutrients, sediment, or metals. One particular management concern for reservoirs is ensuring stormwater hotspots are adequately treated so they do not contaminate drinking water.

Local Provisions:

Table 3.6 Physiographic Factors				
Category	**Stormwater Controls**	**Physiographic Factors**		
		Low Relief	**High Relief**	**Karst**
Bioretention Areas	Bioretention Areas	Several design variations will likely be limited by low head		Use poly-linear or impermeable membrane to seal bottom
Channels	Enhanced Swales	Generally feasible. However, slope <1% may lead to standing water in dry swales	Often infeasible if slopes are 4% or greater	
	Channels, Grass			
	Channels, Open			
Chemical Treatment	Alum Treatment System			
Conveyance System 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
	Detention, Extended Dry			
	Detention, Multi-purpose Areas			
	Detention, Underground			GENERALLY NOT ALLOWED
Filtration	Filter Strips			
	Organic Filters			
	Sand Filters, Surface/Perimeter	Several design variations will likely be limited by low head		Use poly-linear or impermeable membrane to seal bottom
	Sand Filters, Underground			
Hydrodynamic Devices	Gravity (Oil-Grit) Separator			
Infiltration	Infiltration Trenches	Minimum distance to water table of 2 ft	Maximum slope of 6%; trenches must have flat bottom	GENERALLY NOT ALLOWED
Ponds	Wet Pond	Limit maximum normal pool depth to about 4 ft (dugout) Providing pond drain can be problematic	Embankment heights restricted	Require poly or clay liner Max ponding depth Geotechnical tests
	Wet ED Pond			
	Micropool ED Pond			
Porous Surfaces	Modular Porous Paver Systems			
	Porous Concrete			
Proprietary Systems	Proprietary Systems [1]			
Wetlands	Wetlands, Stormwater		Embankment heights restricted	Require poly-liner Geotechnical tests
	Wetlands, Submerged Gravel			

[1] = 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 if used as a primary control.

Table 3.7 Soils

Category	Stormwater Controls	Soils
Bioretention Areas	Bioretention Areas	Clay or silty soils may require pretreatment
Channels	Enhanced Swales	
	Channels, Grass	
	Channels, Open	
Chemical Treatment	Alum Treatment System	
Conveyance System Components	Culverts	
	Energy Dissipation	
	Inlets/Street Gutters	
	Pipe Systems	
Detention	Detention, Dry	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	
Filtration	Filter Strips	
	Organic Filters	
	Sand Filters, Surface/Perimeter	Clay or silty soils may require pretreatment
	Sand Filters, Underground	
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	
Infiltration	Infiltration Trenches	Infiltration rate > 0.5 inch/hr
Ponds	Wet Pond	"A" soils may require pond liner "B" soils may require infiltration testing
	Wet ED Pond	
	Micropool ED Pond	
Porous Surfaces	Modular Porous Paver Systems	Infiltration rate > 0.5 inch/hr
	Porous Concrete	
Proprietary Systems	Proprietary Systems [1]	
Wetlands	Wetlands, Stormwater	"A" soils may require pond liner
	Wetlands, Submerged Gravel	

[1] = 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 if used as a primary control.

Table 3.8 Special Watershed Considerations				
Category	**Stormwater Controls**	**Special Watershed Considerations**		
		High Quality Stream	**Aquifer Protection**	**Reservoir Protection**
Bioretention Areas	Bioretention Areas	Evaluate for stream warming	Needs to be designed with no exfiltration (ie. outflow to groundwater)	
Channels	Enhanced Swales		Hotspot runoff must be adequately treated	Hotspot runoff must be adequately treated
	Channels, Grass			
	Channels, Open			
Chemical Treatment	Alum Treatment System			
Conveyance System Components	Culverts			
	Energy Dissipation			
	Inlets/Street Gutters			
	Pipe Systems			
Detention	Detention, Dry			
	Detention, Extended Dry			
	Detention, Multi-purpose Areas			
	Detention, Underground			
Filtration	Filter Strips			
	Organic Filters			
	Sand Filters, Surface/Perimeter	Evaluate for stream warming	Needs to be designed with no exfiltration (ie. outflow to groundwater)	
	Sand Filters, Underground			
Hydrodynamic Devices	Gravity (Oil-Grit) Separator			
Infiltration	Infiltration Trenches		Maintain safe distance from wells and water table. No hotspot runoff	Maintain safe distance from bedrock and water table. Pretreat runoff
Ponds	Wet Pond	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			
Porous Surfaces	Modular Porous Paver Systems			
	Porous Concrete			
Proprietary Systems	Proprietary Systems [1]			
Re-Use	Rain Barrels			
Wetlands	Wetlands, Stormwater	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			

[1] = 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 if used as a primary control.

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. Table 3.9 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 stormwater control within the regulated area.
- Structural stormwater 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 stormwater structural controls. Consultation with the appropriate regulatory agency is the best strategy.

Local Provisions:

Table 3.9 Location and Permitting Checklist	
Site Feature	**Location and Permitting Guidance**
Jurisdictional Wetland (Waters of the U.S) U.S. Army Corps of Engineers Regulatory Permit	• Jurisdictional wetlands must be delineated prior to siting structural control. • Use of natural wetlands for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater 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 stormwater 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 stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater should be treated prior to discharge into Waters of the U.S. • In-stream ponds for stormwater 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. • Section 401 certification reviews by the Texas Commission on Environmental Quality are required for projects needing a Section 404 Permit.
Water Quality Certification Texas Commission on Environmental Quality (TCEQ)	• TCEQ conducts Section 401 water quality certification reviews of projects requiring a Section 404 permit from the U.S. Army Corps of Engineers for the discharge of dredged or fill material into waters of the U.S., including wetlands. • Specific stream and reservoir buffer requirements. • May be imperviousness limitations • May be specific structural control requirements that may overlap with requirements in this manual. • Mitigation will be required for impacts to existing aquatic and terrestrial habitat.
Impaired Water Bodies Texas Commission on Environmental Quality	• Determine if the project will discharge pollutants of concern into any downstream receiving waters that have been designated as impaired water bodies on TCEQ's *Texas Integrated Report of Surface Water Quality for Clean Water Act Sections 305(b) and 303(d)*. • Stormwater runoff discharges containing pollutants of concern to impaired water bodies will be governed by an entity's Municipal Separate Storm Sewer System (MS4) permit, if applicable.

Table 3.9 Location and Permitting Checklist

Site Feature	Location and Permitting Guidance
Groundwater Management Areas Texas Commission on Environmental Quality	• 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.
Floodplain Areas National Flood Insurance Program / Local Floodplain Administrator	• 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 stormwater 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 stormwater 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 stormwater infiltration. • 50-foot setback for all other structural controls.

3.8.2 Example Application

A 2-mile existing 2 lane roadway is being expanded to a 4 lane divided roadway with a 15 foot median in an urban area within the Dallas/Fort Worth metropolitan area. The roadway will exceed a traffic count of 30,000 vehicles per day. The impervious coverage of the approximate 20 acre site will be 80%. The site drains to two receiving waters, 75% to an urban river with two designated uses on the Texas Surface Water Quality Standards and 25% to an unclassified urban stream. There is a small city park adjacent to the roadway. Low permeability soils limit the use of infiltration practices.

Table 3.10 lists the results of the selection analysis using the screening process described previously. The shaded rows indicate the controls that used alone or in combination may be considered for managing stormwater quality and/or quantity for portions of the site. The **X**'s indicate inadequacies in the control and ✓'s indicate adequate control capabilities for the particular category when considered for this site.

The receiving waters must be evaluated to determine the level of treatment required. The 15 acre area that drains to the urban river will require Level I treatment, while the 5 acre area that drains to the urban stream will require Level II treatment. The level designations are based on the definitions of "Minimal" and "Moderate" receiving water classifications located in Section 3.2.1, Water Quality Treatment Level Criteria, and on Table 3.1, Post-Construction Water Quality Treatment Levels.

There are no special watershed factors or physiographic factors to preclude the use of any of the practices from the structural control list. Other limiting factors of the site might include limited space within the right of way to include non-pipe storm water conveyance necessary for many Level I treatment options; limited space for detention facilities; downstream condition of the urban river and stream; offsite drainage; and large stormwater volumes.

A traditional roadway cross section for the 15 acre roadway section will only require good housekeeping practices such as street sweeping, storm drain inlet cleaning, and proper application of landscape chemicals for Level I treatment as long as the downstream assessment does not show need for additional flood and streambank protection. In order to provide secondary flood control and/or streambank protection for the 15 acres draining to the urban river, a series of grass channels can be placed in the median with the roadway draining towards the median rather than the edges of the right of way. This series of grass channels can be connected to the overall storm drainage system flowing to the urban river. The downstream conveyance system may need to be improved if downstream assessment shows need for additional flood control and/or streambank protection.

Level II treatment for the 5 acre roadway section will require the use of bioretention facilities, an enhanced swale or a detention facility which would all connect to the storm drainage system draining to the urban stream. The additional width of the right of way beyond the roadway limits determines the placement of the bioretention facilities or enhanced swale. These can either be placed in the median or on the edges of the roadway in lieu of curb and gutter with the runoff draining to the location of the stormwater control(s). The dry/extended dry detention pond could be placed in the public park adjacent to the roadway and would be better suited to provide flood control and streambank protection if a downstream assessment shows that they are necessary.

Table 3.10 Sample Structural Control Selection Matrix					
Structural Control Alternative	**Water Quality Treatment Level**	**Streambank Protection and Flood Control**	**Site Applicability**	**Implementation Considerations**	**Other Issues**
Bioretention	Level II	✓[1]	✓[2]	✓	
Enhanced Swale	Level II	✓[1]	✓[2]	✓[3]	
Channels, Grass	Level I	✓[1]	✓[2]	✓[3]	
Dry Detention Pond	Level II	✓	✓	✓[3]	
Extended Dry Detention Pond	Level II	✓	✓	✓[3]	
Filter Strips	Level I	**X**	✓[2]	✓[3]	
Gravity (Oil-Grit) Separator	Level I	**X**	✓[2]	✓	Typically only for drainage areas less than 1 acre
Modular Porous Paver Systems	Level I	**X**	**X**	✓	Not used for travelled lane applications
Porous Concrete	Level I	**X**	**X**	✓	Typically used for low traffic applications
Proprietary Systems[4]	Level I	✓[1]	**UNK**	✓	High cost and maintenance requirements
Scheduled Pollution Prevention Practices	Level I	**X**	**NA**	✓	
Off-Site Pollution Prevention Activities	Level I	**UNK[5]**	**UNK[5]**	**UNK[5]**	

Notes:
1. Only when used with another structural control that provides onsite and downstream flood control
2. Can treat a portion of the site
3. Typically not used in high density / ultra urban settings; however conditions on this site are favorable for this control
4. 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
5. Must be determined by the jurisdiction or agency on a case-by-case basis depending on the type of proposed off-site activity

Additional Local Requirements