THE CITY OF DETROIT
Water and Sewerage Department

Stormwater Management Design Manual

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2018

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1. Introduction

The City of Detroit, led by the Detroit Water and Sewerage Department (DWSD), is taking a more holistic approach to citywide stormwater management. This approach emphasizes the need for consistent, comprehensive stormwater performance standards for both public and private development, while also promoting the use of green stormwater infrastructure (GSI) in Detroit, when technically possible, as the preferred type of stormwater control measures (SCMs). The City of Detroit's Post-Construction Stormwater Management Ordinance (PCSWMO) serves as an important regulatory driver for this improved approach to stormwater management, defining the stormwater performance standards for Detroit. In addition to the PCSWMO, the City of Detroit has updated the Detroit Municipal Code to remove potential barriers to GSI implementation. DWSD provides customers with opportunities lower drainage bills by implement and maintaining eligible GSI practices. These regulatory changes and incentives collectively help ensure that existing development, along with new development and redevelopment projects, create a sustainable city that supports economic growth while protecting Detroiters' public health, water quality, and overall quality of life.

The City of Detroit's Stormwater Management Design Manual (Design Manual) encompasses the engineering methods and technical standards that Detroit's development community and property owners need to plan and build projects that comply with requirements under Detroit's PCSWMO, while ensuring the City meets its state and federal regulatory obligations related to stormwater management.

This chapter provides more information on the intended purpose and audience of this Design Manual, an overview of the benefits of GSI that make this approach the City of Detroit's preferred method for stormwater management whenever possible, and a brief overview of the other chapters included in this Design Manual.

Contents

1.1 Purpose of This Design Manual

In natural, undeveloped landscapes, the hydrologic processes of infiltration of surface water into the ground (both near surface and deep percolation), evaporation, and transpiration work to recycle rainwater through plants and soil, minimizing the transfer of pollutants to surface and ground waters, as shown in Figure 1-1. As land development and urbanization occur, natural or vegetated areas are replaced with streets, parking lots, buildings, and compacted soils. Such impervious surfaces modify the natural hydrology, decrease the permeability of the landscape, and dramatically affect the natural hydrologic cycle.

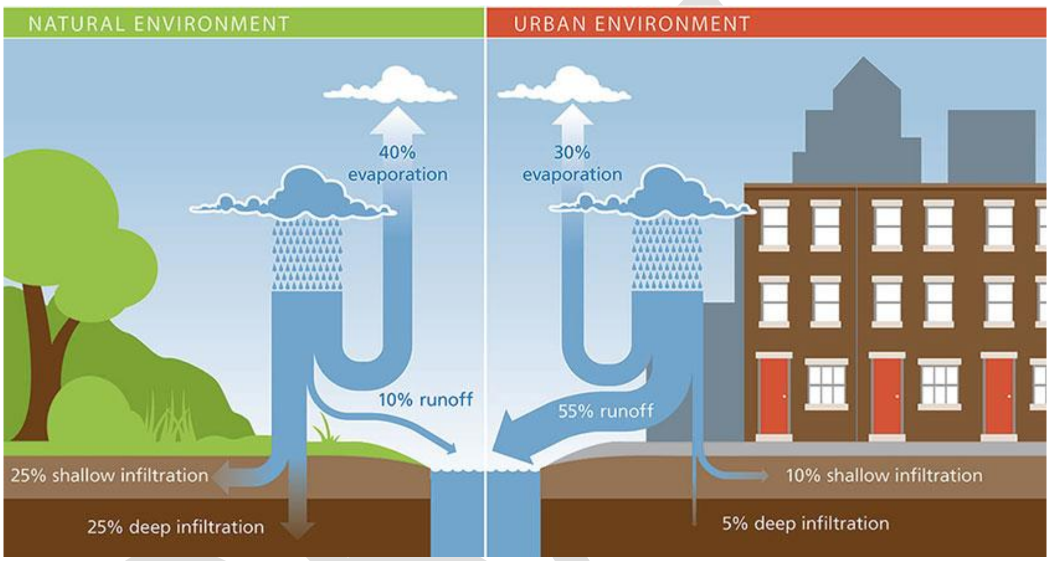


Figure 1-1 Impacts of urbanization on the hydrologic cycle
Source: Philadelphia Water Stormwater Management Guidance Manual

The City of Detroit's sewer system, comprised primarily of combined sanitary and storm sewer lines and a small percentage of separate storm sewer lines, provides drainage of publicly-owned properties, rights-of-way, and private property within the city limits of Detroit. The stormwater draining from these areas should not exceed the system's capacity and be free of excessive pollutants and sediments.

Historical development activities in Detroit have not been subject to stormwater management requirements. This lack of a consistent, proactive approach to managing the stormwater runoff from impervious surfaces to Detroit's sewer system has contributed to overflows of untreated sewage entering the Detroit and Rouge Rivers, as well as localized flooding and basement back-ups, causing public safety risks and property damage.

Combined Sewer Area

A sewer receiving both surface runoff and sewage.

Separate Sewer Area

A sewer which carries storm and surface waters and drainage, but excludes wastewater and industrial waters, other than unpolluted cooling water.

Federal and state regulations and related permits require Detroit to develop programs to address water quality problems caused by discharges from the both combined sewer and separate storm sewer systems. To control both the volume and quality of stormwater discharges, Detroit's PCSWMO requires certain development projects to control the stormwater leaving the property before draining to Detroit's sewer system.

The purpose of this Design Manual is to help Detroit's development community understand how to comply with the PCSWMO requirements and describes the design specifications for controls to meet the required performance standards established to reduce the volume and improve the water quality of stormwater discharges. In addition, the Design Manual is a technical resource for Detroit property owners that choose to implement green stormwater infrastructure practices, stormwater control measures that are eligible for Detroit Water and Sewerage Department (DWSD) drainage charge credits.

The Design Manual provides proven engineering methods for stormwater control measures. It is assumed that the primary users of the Design Manual work within the development community and possess a basic knowledge of hydraulics, hydrology, design, and stormwater management. The Design Manual provides sufficient information and technical resources to:

1) Provide developers and property owners with stormwater management site design guidance to meet Detroit's PCSWMO requirements;
2) Ensure the City of Detroit maintains uniformity in the design standards for SCMs, including GSI practices eligible for drainage credits; and
3) Enable DWSD and other key city departments to conduct effective and efficient design review.

1.2 Considering the Benefits of Green Stormwater Infrastructure While Using This Design Manual

This Design Manual provides developers and engineers with technical information on a variety of SCMs to help comply with the PCSWMO requirements. Although developers and property owners can choose the SCMs that work best for their specific development projects, the City of Detroit encourages developers to consider GSI practices due to the many benefits these types of SCMs can bring to a project, a neighborhood, and Detroit.

As defined in Detroit, GSI practices are SCMs that divert runoff of rain and snowmelt from the sewer system while providing environmental, social, and economic benefits. GSI practices helps control the rate, volume and quality of drainage from impervious surfaces, such as streets, rooftops, and parking lots. These practices also help to maintain and restore natural hydrology by infiltrating, evapotranspiring, capturing, or using stormwater. GSI practices replicate the natural systems that occur at a variety of scales, from single lot to neighborhood to watershed. Common GSI practices at a single lot scale can include rain gardens, bioswales, permeable pavement, infiltration systems and water reuse systems. Larger scale GSI practices can include such amenities as ponds and wetlands.

The City of Detroit will work with the community to maximize appropriate uses of GSI on public and private property. GSI will reduce surface flooding, basement backups, and untreated combined sewer overflow discharges into the Detroit and Rouge Rivers. It will also enhance quality of life, promote public health, increase resiliency, and protect the Great Lakes watershed.

GSI practices help reduce the impact of urbanization on the environment while generating water quality benefits, air quality benefits, social benefits, economic benefits, and aesthetic benefits. GSI practices reduce stormwater discharges to the DWSD sewer system, which results in fewer overflows of untreated sewage entering the Detroit and Rouge Rivers, helping to keep local waterways and the Great Lakes clean for recreation and drinking water. GSI practices also help to create improved habitat for local wildlife, including the City's important migratory birds, and improve urban biodiversity. GSI can bring numerous other benefits to a community, including traffic calming, crime reduction, urban heat reduction, and increased health and well-being, as well as civic pride. These practices can also help to alleviate basement back-ups and localized flooding, reducing property damage and public safety risks.

Widespread installation of GSI practices throughout Detroit can also have a positive economic impact by reducing the City's need to further invest in expensive wastewater treatment facilities, which helps to keep drainage from significantly increasing. At the parcel scale, several GSI practices, as well as removal of impervious cover, are options for reducing drainage bills through DWSD's Drainage Program.

Stormwater Control Measure (SCM)

Any structure, feature or appurtenance that is designed, constructed, operated, practiced, or adopted to reduce the quantity, lower the rate, improve the quality, or otherwise control stormwater runoff through retention, detention, infiltration, reuse, or other stormwater management techniques.

Green Stormwater Infrastructure (GSI)

SCMs that divert runoff of rain and snowmelt from the sewer system while providing environmental, social, and economic benefits. GSI practices helps control the rate, volume and quality of drainage from impervious surfaces and help to maintain and restore natural hydrology by infiltrating, evapotranspiring, capturing, or using stormwater.

Table 1-1 below summarizes the primary benefits of different common GSI practices covered in the Design Manual. Additional information can be found in the Benefits of Green Stormwater Infrastructure on DWSD's Stormwater website.

Table 1-1 Primary Benefits of Common GSI Practices

GSI Practice	Water Quantity and Quality Benefits	Habitat and Wildlife Benefits	Community Benefits	Educational Benefits	Economic Benefits
Bioretention and Rain Gardens	💧	💧	💧	💧	💧
Cisterns	💧			💧	💧
Green Roofs	💧	💧	💧	💧	💧
Permeable Pavements	💧			💧	💧
Trees and Green Space	💧	💧	💧	💧	💧
Remove Impervious Cover	💧				💧

1.3 Chapter Overview of This Design Manual

The descriptions below provide an overview of each chapter in this Design Manual.

Chapter 2 – Regulatory Requirements

This chapter describes the regulatory requirements and other programmatic drivers for stormwater management in Detroit, with emphasis on the PSCMO requirements related to water quality, channel protection, and flood control.

Chapter 3 – Site Design and Stormwater Management

This chapter presents guidelines and considerations for designing site development projects including site assessment, site and landscape design principles, and preliminary concept development. The chapter also illustrates how to integrate stormwater management components into site designs for a variety of building sites, open spaces, and building types.

Chapter 4 – Hydrologic Procedures

This chapter provides precipitation data, as well as acceptable methods for calculating runoff volumes and peak discharge rates.

Chapter 5 – Drainage Conveyance

This chapter provides standards and requirements for the design of storm sewer systems to ensure consistency with the current requirements for the City's public roadways and ensure the safe and effective flow of stormwater through conveyance systems that are part of the site design.

Soil infiltration test

Chapter 6 – Soil, Aggregates and Water

This chapter contains general information on the physical properties of soil and aggregates, with a focus on how water moves through these materials and the need for geotechnical information to support the design and construction of stormwater control measures, particularly GSI practices intended to promote infiltration.

Retention basin with outlet riser

Chapter 7 – Large Detention Practices

This chapter discusses the different types of surface and subsurface detention practices, including basic detention basins, extended dry detention, and extended wet detention, and summarizes technical information necessary to design, construct, and maintain these stormwater control measures.

Bioretention between road and sidewalk

Chapter 8 – Bioretention

This chapter introduces bioretention practices, including bioswales and tree box filters, and summarizes the technical information for design, construction, and maintenance. Bioretention is a very flexible practice that can be used in a variety of settings and is the most common GSI practice.

Chapter 9 – Infiltration Basins and Trenches

This chapter covers the technical information for designing, constructing, and maintaining infiltration basins and trenches. Infiltration basins and trenches are designed to encourage percolation and ground water recharge of stormwater runoff. Infiltration basins are typically larger shallow surface impoundments used to manage stormwater runoff from areas between 5-50 acres while infiltration trenches are narrow, linear practices that are used to manage stormwater runoff from area less than 5 acres, like along a roadway or parking lot.

Chapter 10 – Permeable Pavement

Permeable concrete pavers

This chapter summarizes the information for designing, constructing, and maintaining several types of permeable pavement, including porous asphalt, pervious concrete, pervious pavers, and grid pavement systems. Permeable pavement allows streets, parking lots, sidewalks and other impervious covers to retain the infiltration capacity of underlying soils while maintaining the structural and functional integrity of traditional pavements.

Chapter 11 – Water Harvesting

Detroit Farm and Garden cistern

This chapter summarizes the information for designing, constructing, and maintaining water harvesting practices such as cisterns. Water harvesting is a practice that captures stormwater runoff often from rooftops for later use as irrigation or alternative grey water uses between storms, providing a potential water bill savings. Cisterns are larger systems (up to 10,000 gallons or even larger) that are more often used on commercial or industrial sites and can be placed aboveground or below ground.

Chapter 12 – Green Roofs and Walls

Green roof

This chapter summarizes the information for designing, constructing, and maintaining green roofs and walls that capture rainfall in a layer of vegetation and growing media, with excess rainwater directed to roof drains and downspouts.

Chapter 13 – Stormwater Wetlands

Stormwater wetland

This chapter summarizes the information for designing, constructing, and maintaining stormwater wetlands, shallow-water ecosystems designed to treat stormwater runoff in low-lying areas or along river corridors where water tables are high.

Chapter 14 – Manufactured Treatment Systems

This chapter describes the DWSD review and approval process for proprietary manufactured treatment systems. Manufactured treatment relies on a variety of mechanisms to remove pollutants

Silt fence at a construction site

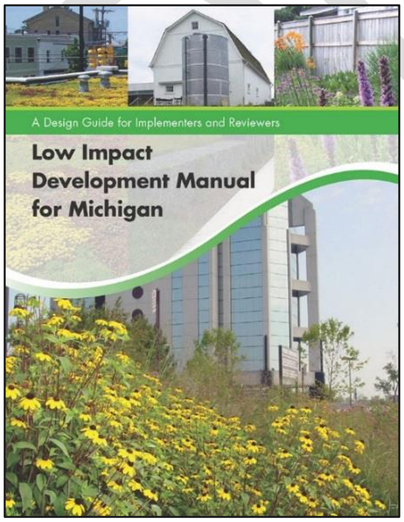
such as sediment, trash, and floatable debris, from stormwater runoff. Two common types of manufactured treatment devices include hydrodynamic separators which use chambers to trap sediment and filtering systems which use a settling chamber then filter to remove specific pollutants.

Chapter 15 – Construction Site Soil Erosion and Sediment Control

This chapter provides information on state and county requirements related to soil erosion and sediment control (SESC) at active construction sites that disturb greater than one acre or are within 500 ft of a lake or stream. Construction site operators must obtain an SESC permit and develop a SESC plan, which includes measures such as silt fence, stabilized construction exits, and other practices that will be used to minimize erosion and off-site sedimentation.

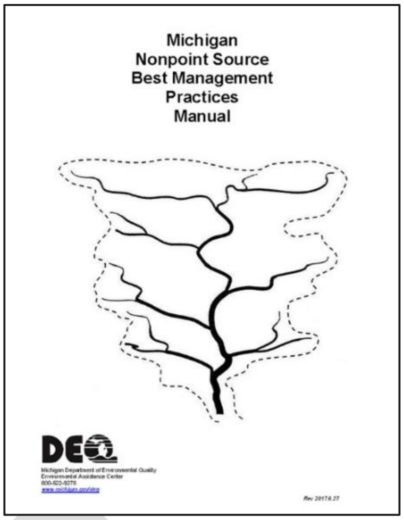
1.4 Additional References

Although this Design Manual is the only technical resource crafted specifically for Detroit's PCSWMO compliance, there are other stormwater management resources with a statewide perspective that may interest the development community and property owners in Detroit interested in effective stormwater management.



Southeast Michigan Council of Governments (SEMCOG) ***Low Impact Development Manual (LID) for Michigan: A Design Guide for Implementers and Reviewer*** **(2008).** This manual provides guidance on how to apply low impact development (LID) to new, existing, and redevelopment sites. It contains technical details of stormwater controls but also provides a large scope of managing stormwater through policy decisions. The information contained within the manual is informational only and should not be construed as requirements for Detroit's PCSWMO. An electronic copy of the Michigan LID Manual can be found on the SEMCOG website, www.semcog.org.

MDEQ Michigan Nonpoint Source Best Management Practices (BMP) Manual. This manual provides BMP design guidance to address minimizing the amount of stormwater management and/or treatment required as well as stormwater infiltration practices and natural channel design. An electronic copy of the DEQ Guidebook of BMPs can be found on the MDEQ website, *http://www.michigan.gov/deq/0,4561,7-135-3313_71618_3682_3714-118554--,00.html*



2. Regulatory Requirements

The City of Detroit's Post-Construction Stormwater Management Ordinance (PCSWMO), administered by the Detroit Water and Sewerage Department (DWSD), is designed to prevent combined sewer overflows (CSOs), decrease the discharge of polluted runoff into area waterways, prevent localized flooding, and increase the infiltration of runoff into groundwater systems to emulate more natural conditions. This chapter provides guidance for complying with the City's PCSWMO, as well as other stormwater management requirements from non-City regulating authorities.

Contents

2.1 Detroit Post-Construction Stormwater Management Ordinance

The City of Detroit is committed to responsibly managing stormwater runoff in a way that protects existing infrastructure, the environment, and both public and private property. DWSD manages a variety of stormwater management programs to aid in the reduction of stormwater entering Detroit's combined and separate storm sewer systems. These programs include the DWSD Drainage Charge Program for public and private property, the DWSD Green Stormwater Infrastructure Program for public property, and Detroit's PCSWMO, which is included in Chapter 56 Article 4 of the Detroit Municipal Code (the Code). DWSD is responsible for administering this ordinance as a part of the City's overall approach to stormwater management. More information on these DWSD stormwater management programs is available at http://www.detroitmi.gov/stormwater.

2.1.1 PCSWMO Applicability

Any development site with regulated construction activity that involves the replacement or creation of one-half acre (21,780 square feet) or more of impervious surface is subject to the PCSWMO. DWSD may also require that any construction activity meeting certain conditions (as specified in the Code) must also comply with these requirements. Additional details regarding the applicability threshold can be found in Section 56-3-101 of the Code.

Exemptions

There are limited exemptions from the PCSWMO (see Detroit Municipal Code Section 56-3-102):

- The improvement or construction of an individual Single Family Detached Dwelling as defined in the City Zoning Ordinance;
- Emergency maintenance work performed for the protection of public health and safety; and
- A regulated construction activity that discharges stormwater directly to the Detroit River or Rouge River via a conveyance not owned by DWSD.

Figure 2-1 illustrates how to determine whether a development site needs to comply with PCSWMO requirements.

Construction Activity

A human-made activity, including without limitation clearing, grading, excavating, construction and paving, that results in a change in the existing cover or topography of land, including any external demolition, modification or alteration of a site or the footprint of a building. Does not include re-surfacing of an asphalt, concrete or similar parking lot that does not expose the subgrade.

Regulated Construction Activity

Construction Activity that is subject to the provisions of the regulations. A Regulated Construction Activity may occupy all or part of a Development Site.

Impervious Surface

Any surface area that prevents or substantially impedes the entry of water into the soil in a manner that such water entered the soil under natural conditions pre-existent to development, or which cause water to run off the surface in greater quantities or at an increased rate of flow than that present under natural conditions pre-existent of development, including but not limited to roofs, parking lots, compacted gravel and dirt, driveways, sidewalks and storage areas. (*Detroit Municipal Code Section 56-3-2*)

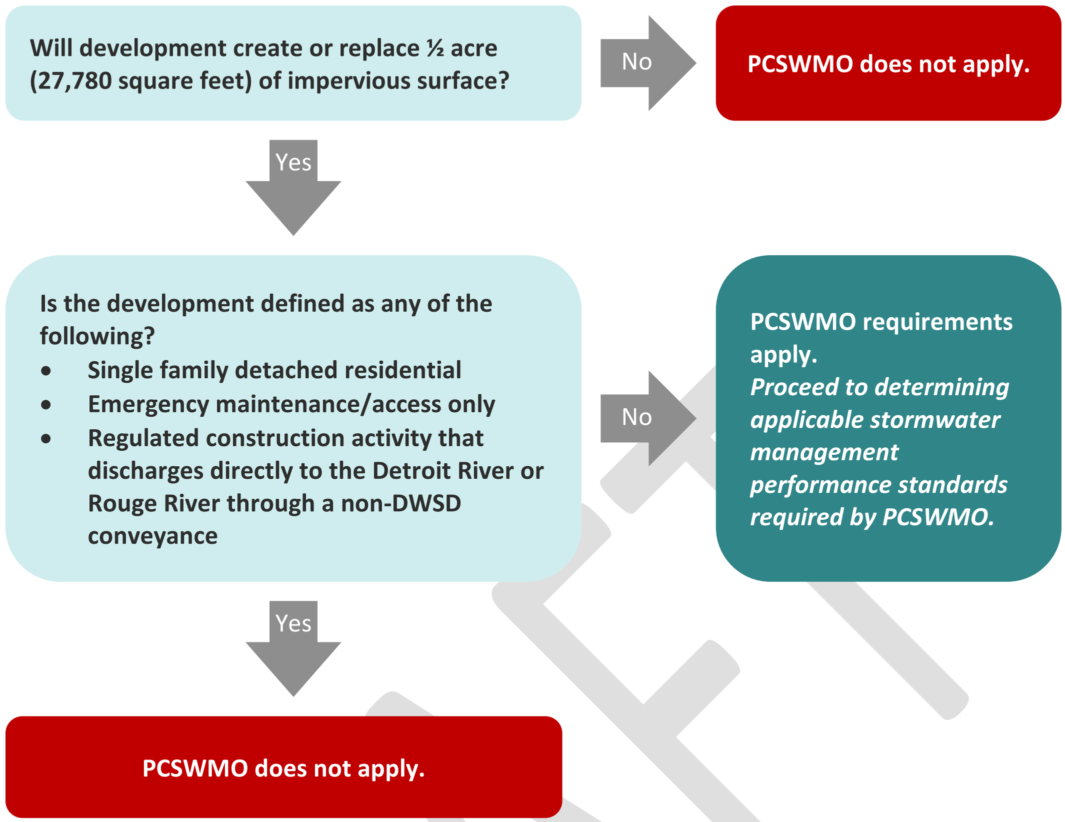


Figure 2-1 Determining PCSWMO Applicability

2.1.2 Stormwater Management Performance Standards and Applicability

The PCSWMO requires the use of SCMs to manage stormwater on a development site and stipulates specific performance standards that the SCMs must meet to be compliant. The performance standards contained in the PCSWMO have four main objectives:

- Protect water quality by removing pollutants in stormwater or preventing combined sewer overflows
- Protect the channels of receiving streams
- Protect the integrity of DWSD's infrastructure
- Reduce the potential for localized flooding due to DWSD infrastructure limitations

This is accomplished by requiring SCMs that retain, detain and treat the runoff which falls onto a development site prior to it discharging off the property. The standards specify the volume that shall be retained permanently on-site (e.g. infiltrated into the ground) and treated, the volume that must be detained temporarily, and the rate at which that detained water can eventually be conveyed to DWSD's sewer system.

The applicable stormwater management performance standards for a project depend upon the discharge location. Table 2-1 summarizes the performance standards by discharge location. A project subject to PCSWMO requirements may discharge to the combined sewer system or the separate sewer system owned and operated by DWSD. The vast majority of Detroit is served by the combined sewer area. Only some of the fringe areas immediately adjacent to the Rouge River, the Detroit River, and the canals may be served by a separate sewer system. However, there are some limited areas where DWSD is separating the combined sewer area. To verify if a project location discharges to a combined sewer area or to a separate storm sewer area, contact DWSD.

Meeting the performance standards is dependent on managing stormwater from the regulated area of a development site. The regulated area is the portion of a development site which is used to calculate the stormwater which need to be managed to meet the performance standards. This area varies based on the location of the development site as well as the amount of construction activity disturbance. The associated regulated area for each stormwater performance standard is defined in Table 2-1 and the standard-specific discussions below. The methodologies for using regulated area to calculate a project's stormwater management volume and flow requirements for a project are discussed in Chapter 4.

Regulated Area

The portion of a Development Site used as the basis to determine compliance with the performance standards set forth in the PCSWMO.

Table 2-1 Stormwater management performance standards and regulated area by discharge location

<table>
<tr><th>Discharge To</th><th>Water Quality</th><th>Infrastructure / Channel Protection</th><th>Flood Control</th></tr>
<tr><td>Combined Sewer System
(97% of the Detroit System)</td><td>Match natural conditions for peak flow and volume for the 90th percentile storm event

Regulated Area: Construction activity < 50 percent of development site = construction activity area

Construction activity > 50 percent of development site = entire development site</td><td>The peak flow rate of stormwater runoff shall not exceed the predevelopment peak flow rate for the 2-year storm.

Regulated Area: Entire development site</td><td rowspan="2">*Peak flow only

Drainage Area < 5 acres, manage 10-year storm

Drainage Area > 5 acres, manage 100-year storm

Release rate of 0.15 cfs/acre

Regulated Area: Entire development site</td></tr>
<tr><td>Separate Sewer System</td><td>Peak flow and volume

Match natural conditions for peak flow and volume for the 90th percentile storm event

Regulated Area: Regulated construction activity</td><td>*Peak flow and volume

Match natural conditions for peak flow and volume for the 2-year, 24-hour storm event

Regulated Area: Regulated construction activity</td></tr>
</table>

*Discharge via a DWSD owned separate storm sewer system to Detroit River or Rouge River downstream of the Rouge Turning Basin are not subject to Channel Protection and Flood Control performance standards.

Local Flood Control Standards

All projects that are regulated by the PCSWMO must account for local flood control. The local drainage system has limited capacity and may be overwhelmed by large storm events. Therefore, to minimize local flooding, development projects must detain stormwater runoff during large runoff events and discharge the runoff off-site at a slower rate. Peak flow rates from large storm events shall be temporarily detained as a part of site development, and released at a controlled rate. Table 2-2 summarizes the performance standards that apply for local flood control citywide, depending on the size of the drainage area. The entire development site is the regulated area for flood control performance standards.

Drainage Area

The land area from which stormwater runoff drains to a common point, including any area lying beyond the boundaries of a Development Site.

Table 2-2 Local Flood Control Performance Standards

Design Event	Criteria	Applies
10-year, 24-hr	Peak flow <= 0.15 cfs/acre	Drainage area < 5 acres
100-year, 24-hr	Peak flow <= 0.15 cfs/acre	Drainage areas >= 5 acres

Combined Sewer Area Performance Standards

Water Quality

Water quality performance standards help protect the combined sewer infrastructure from sediment accumulation and reduces stormwater treatment needs at the wastewater treatment plant and wet weather facilities. The water quality performance standards require the removal of sediment from stormwater runoff in two ways – either through treatment of the runoff prior to discharge or through the retention of stormwater on-site. Often, meeting the retention standard on a project site will also meet the treatment standard as well. If not, however, additional treatment of the stormwater prior to discharge may be necessary.

The regulated area for the water quality performance standard for projects that discharge to the combined sewer system is determined based on the percentage of the development site disturbed by construction activities, as shown in Figure 2-2. If construction activities will cover less than 50 percent of the development site then the regulated area is equal to the area of construction activity. However, if construction activities will cover 50 percent or more of the development site, then the regulated area is the entire development site as shown in the right box of Figure 2-2.

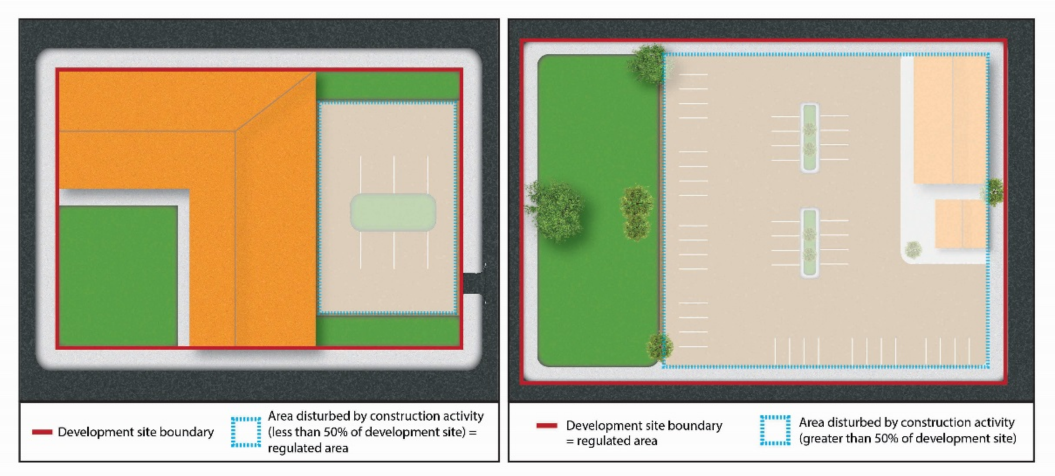


Figure 2-2 Regulated area for water quality performance standard in combined sewer area

<u>Treatment</u>

In combined sewer areas regulated construction activities must treat the 90-percent annual non-exceedance storm. This water quality volume must be treated to (1) remove 80 percent of the total suspended solids (TSS) projected to be in uncontrolled runoff from the site in the post-construction condition or (2) have an effluent concentration of less than or equal to 80 mg/L TSS.

Water Quality Volume

The volume of stormwater runoff generated by the 90th percentile storm over the Regulated Area of a Development Site.

Natural Condition

The condition of land that is predominantly covered in vegetation that is sustainable without regular human maintenance, such as irrigation, mowing, or fertilization. Examples of natural cover include forest, woodland, meadow, grassland, or shrubland.

<u>Retention</u>

All regulated construction activities within the combined sewer area must ensure that the runoff volume and peak flow rate of stormwater runoff leaving the regulated area of the site post-construction do not exceed the runoff volume and peak flow rate for the water quality volume under natural conditions.

Infrastructure Protection Standards

In addition to the citywide flood control standards, the City has established more focused infrastructure protection standards which limit peak flows to prevent combined sewer overflows and flooding. The peak flow rate of stormwater runoff shall not exceed the predevelopment peak flow rate for the 2-year storm. The entire development site is the regulated area for complying with the combined sewer area infrastructure protection standards.

Separate Storm Sewer Area Performance Standards

Water Quality

Separate storm sewers discharge into receiving waters – streams, rivers, wetlands – without treatment before discharge. To protect the water quality in these waterbodies, all regulated construction activities which discharge to the separate storm sewer must provide water quality treatment and control peak flow rates to provide channel protection. The regulated construction activity is the regulated area for purposes of complying with the water quality performance standard for separate sewer areas.

In separate storm sewer areas regulated construction activities must treat the 90th percentile annual non-exceedance storm. This water quality volume must be treated to (1) remove 80 percent of the total suspended solids (TSS) projected to be in uncontrolled runoff from the site in the post-construction condition or (2) have an effluent concentration of less than or equal to 80 mg/L TSS.

Channel Protection

For channel protection of local streams, the runoff volume and peak flow rate of stormwater runoff leaving the regulated area post-construction must not exceed the runoff volume and peak flow rate which would occur under natural conditions for all storms up to and including the 2-year, 24-hour storm event. Discharges from that drain into any portion of the City's storm sewer discharging directly to the Detroit River or downstream of the Rouge River Turning Basin are exempt. The regulated area is the regulated construction activity for complying with the channel protection performance standards.

Alternative Compliance

Per Section 56-3-106 of the Detroit Municipal Code, applicants may apply for alternative compliance from water quality volume retention requirements (found at Section 56-3-105(d)(1)(c)) if the applicant demonstrates the presence of extraordinarily difficult site conditions that make retaining water onsite infeasible. Alternative compliance may be granted for up to 100 percent of the required volume.

Alternative compliance may be accomplished either through offsite mitigation or paying an in-lieu fee. *Offsite mitigation* involves constructing a stormwater practice elsewhere in the city on land owned by the applicant or on land owned by the city (with permission from the city). *In-lieu fee* involves paying a one-time fee to the City in lieu of constructing stormwater management practice.

Extraordinarily Difficult Site Conditions

Extraordinarily Difficult Site Conditions can include, but are not limited to, one or more of the following:

(1) **Sub-surface condition limiting infiltration**. The presence of sub-surface conditions, including soil contamination or shallow depth to bedrock or groundwater, that

present significant and atypical technical requirements for mitigation, stormwater management measure design or installation, or create a likelihood for subsurface pollutant flume transport; or

(2) **Unique conditions that would require substantial re-grading for stormwater controls**. Unique topographic or geologic conditions that would require site re-grading or re-contouring substantially different from typical and customary practices for the installation of Stormwater Control Measures; or

(3) **Potential for off-site basement flooding**. Surface or subsurface conditions indicating a likelihood that basement flooding on properties other than the Development Site are reasonably foreseeable if Stormwater Control Measures are installed; or

(4) **Conditions that require pumping of stormwater**. Unique site or operational conditions that would require pumping or other mechanical routing of stormwater to meet the performance standards of Section 56-3-105(d)(1)(c); or

(5) **Other conditions**. Other conditions that, in the judgment of the Department, present a substantial barrier to the safe and effective construction or operation of Stormwater Control Measures.

Offsite Mitigation Requirements

Offsite mitigation projects must meet the following conditions:

1. A post-construction stormwater management plan shall be prepared for the offsite mitigation project.
2. In all cases, land rights, access agreements or easements, and a maintenance agreement and plan shall be provided to ensure long-term maintenance of any off-site mitigation project prior to approval of the off-site mitigation proposal.
3. Installation of the off-site mitigation project shall be completed: (a) within two (2) years from the date that the stormwater management design plan is approved, or (b) prior to full completion of the development project related to the off-site mitigation project, whichever of (a) or (b) is earlier.
4. All requirements for on-site stormwater management shall also apply to off-site mitigation projects. These requirements include but are not limited to a stormwater management design plan, inspections, maintenance, and performance bonds.

In-lieu Fee

Applicants approved for alternative compliance may pay a one-time fee in accordance with the following formula

*Required retention volume (gallons) * Unit Cost ($/gallon)*

The Unit Cost is a monetary amount established by a resolution of the DWSD Board of Water Commissioners based on the average cost per gallon of runoff for DWSD to construct green stormwater infrastructure projects on City properties. A list of the most current unit cost is maintained by DWSD. Unit costs can change each year. Obtain the latest from DWSD.

Alternative Compliance Requests

DWSD will consider alternative compliance requests only after an applicant has demonstrated that the conditions cannot be overcome or mitigated through reasonable redesign, or without substantial interference with the present or intended use of the Development Site.

The alternative compliance process is time consuming and requests should be submitted as early as possible in the development of the project. Applicants must submit alternative compliance requests to DWSD with supporting documentation, including the hydrological analysis as described in Chapter 4, site plans, geotechnical reports, environmental site assessments, and engineering analysis. Requests for alternative compliance without supporting documentation will not be considered.

2.1.3 Buffers

A buffer with a minimum width of 25 feet must be established and maintained, or preserved, along the edge of any surface water and any regulated wetland, as shown in Figure 2-4.

Figure 2-4. Required 25 ft. buffer

2.1.4 Post-Construction Stormwater Management Plan

The Post-Construction Stormwater Management Plan (PCSWMP) documents compliance with the PCSMO requirements for a project. The PCSWMP is submitted as a part of an applicant's site plan. DWSD reviews and approves all PCSWMPs; approval is necessary prior to the approval of a site plan by the City.

Contents of a PCSWMP

The required components for a PCSWMP are as follows:

(1) A map which shows the discharge location(s) for all post-construction stormwater runoff which will leave the Development Site, and the boundaries of the drainage area tributary to each discharge location.

(2) A map which shows the boundaries of the Development Site, Common Plan of Development if applicable, and the Regulated Construction Activity, clearly indicating areas of disturbance, the boundaries of any no-build or non-disturbance areas, all points of egress from the Development Site to a public right-of-way, and all easements and other encumbrances.

(3) The preparation and property recording of all required easements, deed restrictions, reservation of rights-of-way, or other protective covenants to ensure sufficient access for the purposes of maintenance, inspection, operation, repair, or replacement of any installed stormwater control measures. The Applicant must also ensure that any future modification of the site is consistent with the approved PCSWMP. See Section 56-3-110 and 56-3-111 of the City Municipal Code for additional information on easement recording requirements.

(4) The required calculations establishing compliance with the Post Construction Stormwater Management Performance Standards set forth in Section 48-3-105 this Division.

(5) The design specifications and calculations, construction details, and locations for all proposed Stormwater Control Measures, whether located on the Development Site or elsewhere, as selected and designed using the guidance and required standards in the Stormwater Management Design Manual.

(6) A map which shows the locations and descriptions of all access drives easements necessary to allow for construction, inspection, operation and maintenance of all proposed Stormwater Control Measures.

(7) An Operation and Maintenance Plan prepared by a professional engineer or landscape architect properly licensed to practice in the State of Michigan containing maintenance requirements and protocols for each selected SCM; a schedule of inspection and maintenance activities, and procedures and checklists for each SCM consistent with the provisions for each selected SCM in this Design Manual; a signed certification statement accepting responsibility for the operation, maintenance and inspection of the SCM.

(8) A copy of all applicable state and federal permits related to erosion, water resource and stormwater management for the Regulated Project.

Applicants should complete DWSD's PCSWMP application form and checklist, available in Appendix A of this Design Manual, and submit all required documentation to DWSD for review and approval. Both the PCSMP and the associated O&M Plan must be developed or sealed by a professional engineer or landscape architect properly licensed to practice in the State of Michigan.

2.2 Other City Requirements

In addition to the PCSWMO, it is important to keep in mind that other City requirements and permits may apply to the development site. For example, DWSD issues sewer tap permits and BSEED issues plumbing permits for private property connections to the City sewer. Changes to parking area configuration for purposes of stormwater management will require alignment with the City's zoning requirements.

The applicant shall ensure that all drainage related City permits are obtained.

If sewer tap permits are necessary, contact DWSD at 313-964-9236.

Plumbing permits for private property connections to City sewer, contact BSEED's plumbing inspector at 313-224-3158.

Confirm the City's parking lot requirements with BSEED's Zoning Department at 313-224-1317.

2.3 Special Use Areas and Conditions

2.3.1 Natural Wetlands

In watersheds with a lot of development pressures and/or sensitive wetland communities, a major objective should be to protect wetlands from upstream stormwater impacts. Sensitive wetlands are affected by even small changes in inundation and water quality; therefore, special stormwater criteria may be needed when working near a wetland or within its contributing drainage area (CDA). The stormwater management strategies that have typically been used to protect wetlands range from merely requiring pretreatment of the discharges into the wetland to excluding 100 percent of all new discharges into a wetland.

This section applies to natural wetlands (as opposed to stormwater wetland systems that are constructed exclusively for stormwater management purposes). When a natural wetland is incorporated into the overall stormwater management design, the following requirements apply:

- Obtain all necessary wetland permit approvals from the Michigan Department of Environmental Quality (MDEQ) and City before approval of the final site plan.
- Protect natural wetlands from adverse changes in runoff quality and quantity associated with any land developments/redevelopments.
- Prohibit direct discharge of untreated stormwater to a natural wetland. Prior to discharge to the wetland, pre-treat all runoff from the development to remove sediment and other pollutants. Construct treatment facilities before property grading begins. Clean and stabilize all basins prior to final acceptance.
- Site drainage patterns will not be altered in any way that will modify existing water levels in protected wetlands. Any alteration of drainage patterns that may affect water levels require prior approval and all applicable permits from the MDEQ and/or DWSD.

MDEQ offers a Wetland Identification Program (WIP) to assist the public in identifying wetland and non-wetland (i.e., upland) areas on their property. Wetland Inventory Maps are also available on the MDEQ website. Since the maps are not based upon an on-site review, the DEQ does not provide a jurisdictional guarantee or a map specific to the parcel. The MDEQ Wetlands Map Viewer also provides a Status and Trends Tool which allows users to compare historic wetland data to current information.

- A qualified professional with specific wetland expertise will oversee wetland construction, reconstruction, or modification.
- Maintain or restore a permanent buffer strip, vegetated with native plant species around the periphery of wetlands.
- Protect wetlands during construction by appropriate soil erosion and sediment control measures.

For projects subject to jurisdiction under the Wetlands Protection Act, the applicant shall demonstrate to the issuing authority that the discharge velocities will not cause erosion or scouring at, or downstream of, the point of discharge.

2.3.2 Floodplains

Floodplain Regulatory Authority found in Part 31, Water Resources Protection, of the Natural Resources and Environmental Protection Act (NREPA) states a permit is required from MDEQ, Water Resources Division, for any occupation, construction, filling, or grade change within the 100-year floodplain of a river, stream, drain, or inland lake. Bridges and culverts are considered an occupation of the floodplain, as are activities that involve storage of materials in the floodplain. Any earth disturbance over one-acre (43,560 square feet) in size or within 500 feet of a lake or stream needs a soil erosion and sedimentation control (SESC) permit. Two permits, described below, are required. Obtain all necessary floodplain permit approvals from the MDEQ and provide elevation certificate for structures to BSEED before approval of the final site plan.

- For the 100-year floodplain, the applicant must demonstrate that development in a FEMA Special Flood Hazard Area (SFHA) floodplain does not increase the 100-year flood elevation. This must be supported by calculations or computer model output that demonstrates the pre-development and post-development flood elevations. The applicant should include an SFHA permit and the appropriate fee with the Stormwater Management Plan. Floodplain Regulatory Authority found in Part 31, Water Resources Protection, of the NREPA A permit is required from MDEQ, Water Resources Division, for any occupation, construction, filling, or grade change within the 100-year floodplain of a river, stream, drain, or inland lake. Bridges and culverts are considered an occupation of the floodplain, as are activities that involve storage of materials in the floodplain.
- The applicant must demonstrate that any development within a 100-year floodplain will not diminish flood storage capacity. Compensating storage is required for all lost floodplain storage. The applicant must demonstrate that any volume of fill placed in the 100-year floodplain is compensated with an equal volume of material removed above the ordinary high-water table and below the 100-year flood elevation. Volume calculations are required for the compensating storage.

2.4 Construction Site Soil Erosion and Sediment Control

The City of Detroit's soil erosion and sedimentation control (SESC) program is administered by Wayne County Department of Public Services – Environmental Services Group. Projects which meet the county's threshold requirements must apply for a Wayne County SESC permit before any earthwork can begin.

Information on Wayne County's SESC program, including information on the permit and application kit, is available at: http://www.waynecounty.com/doe/soilerosion.htm

In addition, projects in Detroit which have earth disturbance greater than 1 acre and have a point source discharge of stormwater to waters of the state (either directly or through a separate storm sewer) are also required to follow MDEQ's Permit-by-Rule. *Stormwater discharges to Detroit's combined sewers are not required to follow the MDEQ Permit-by-Rule.*

Information on MDEQ's SESC program and permits, including training information and guidance documents, is available at: http://www.michigan.gov/deq/0,4561,7-135-3311_4113---,00.html

Coverage under this Permit-by-Rule is automatic for MDEQ regulated projects which disturb more than one but less than five acres of land as long as applicants have a Wayne County SESC permit and a certified storm water operator inspects the site weekly and within 24 hours of a rain event resulting in a discharge of storm water from the site. However, MDEQ regulated projects which disturb more than five acres of land must apply for Permit-by-Rule coverage by submitting a Notice of Coverage (NOC) to the MDEQ Water Resources Division. Permit-by-Rule requirements and NOC applications can be found on the MDEQ website.

2.5 References

MDEQ. 2014. Post-Construction Stormwater Runoff Controls Program Compliance Assistance Document. MS4 Program, Water Resources Division, Michigan Department of Environmental Quality, Lansing.

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2.5 References

MDEQ. 2014. Post-Construction Stormwater Runoff Controls Program Compliance Assistance Document. MS4 Program, Water Resources Division, Michigan Department of Environmental Quality, Lansing.

3. Site Design and Stormwater Management

Development in Detroit can range from reuse of a previously demolished site, to reuse of an existing building with changes to the site, to new demolition and new construction. Stormwater management opportunities for these different types of development projects will vary, depending on site-specific conditions. Regardless of the type of development, the site design process for the project should integrate stormwater management considerations at the outset. This chapter presents guidelines and considerations for designing site development projects including data requirements, agency coordination, site design principles, and system components. The chapter also illustrates how site designs for a variety of building sites and types, as well as open spaces, can integrate SCMs.

Contents

3.1 Integrated Site Design Approach

The recommended approach to site design is one that considers stormwater management in the initial stages of the design process and continually integrates these considerations at every decision point. This process works sequentially to first reduce the quantity of stormwater runoff to be managed, and then to iteratively determine the most effective and cost efficient means for designing structural and non-structural SCMs, including GSI practices, to meet the applicable PCSMO requirements, as well as other potential project goals such as drainage credits and water conservation through GSI practices. Integrated site design early in a project increases the potential for identifying SCMs that can provide multiple benefits, such as those associated with GSI discussed in Chapter 1. **Error! Reference source not found.** provides a consolidated work flow process of the integrated site design approach that serves as an organizing framework for this chapter.

Identify all requirements necessary for the project and determine project goals and objectives

↓

Perform the site assessment and map key opportunities and constraints

↓

Begin design by looking for ways to reduce impervious surfaces and reduce stormwater management needs

↓

Review options for additional stormwater control measures

↓

Layout conceptual design for the site using design principles, perform calculations, and iterate to a suitable design

↓

Begin final design and assembly of the Post-Construction Stormwater Management Plan

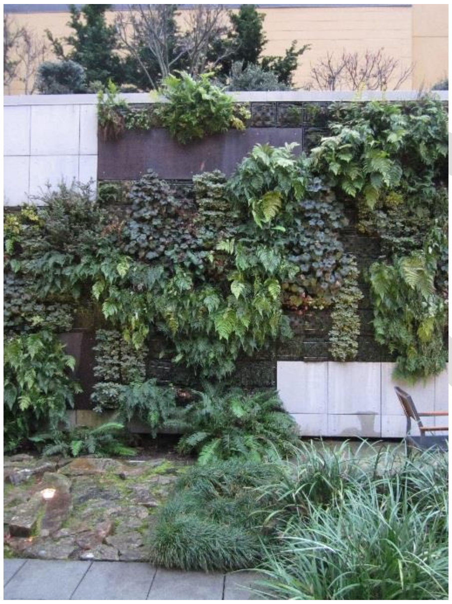
Figure 3-1 Integrated Site Design Process

Stormwater as a Site Amenity

Historically, stormwater has been treated as a nuisance that carries contaminants, floods basements and streets, and needs to be conveyed off site as quickly as possible. However, when stormwater is managed at the source, it can serve as a valuable resource for groundwater recharge, irrigation for plants, and even be stored and reused for non-potable functions, creating potential long-term cost savings. Using reclaimed stormwater for irrigation, toilet flushing or other applications reduces or eliminates the need to purchase potable water for these uses. Furthermore, contemporary design philosophies have emerged which aim to maximize the multitude of benefits that property owners, residents, and communities can receive when working with stormwater specifically as a design element. Many of the design principles presented throughout this chapter aid in maximizing the benefits of treating stormwater as an amenity.

3.2 Identify Regulatory Requirements and Determine Project Goals and Objectives

The first step in the integrated design approach is to identify and review regulatory requirements and, based on the understanding of the applicable requirements, determine the project goals and objectives.



A "Living Wall" and bioretention area in Portland, OR.
Source:

3.2.1 Identify all necessary regulatory requirements

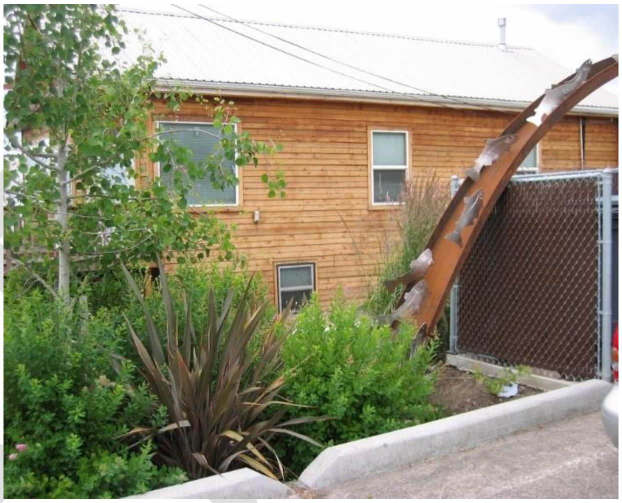
A thorough understanding of all the applicable local, state and federal requirements for the proposed project site will define the applicable performance standards, the necessary permits, and the review and approval processes. Knowing this information upfront can reduce project delays, and in some cases, change the trajectory of the project to avoid the need for the permit all together. Carefully review the regulatory requirements, including stormwater performance standards and associated regulated area definitions, in Chapter 2 of this Design Manual prior to proceeding with design.

3.2.2 Define goals for the project

In large part, the applicable regulatory requirements will define the goals and objectives for a project, but there may be additional goals that a project team hopes to achieve. For example, some project teams may want to incorporate GSI practices to reap the additional benefits, such as increased property values and decreased drainage bills, while others may want to include other elements that may lead to grant funding. Understanding the possibilities and defining the projects goals early makes the goals much more likely to be carried out during the design process.

3.3 Perform the Site Assessment

A thorough site assessment is a key next step of the design process. The site assessment consists of two phases: an inventory of the site, where the physical, biological and cultural attributes are identified; and the analysis, which determines where the constraints and opportunities exist and how they will guide the selection of the most appropriate stormwater management approach and design for the site. This section serves as a guide for completing a thorough site assessment.



Artistic downspout into a bioretention area (Portland, OR)
Source:

3.3.1 Conducting a Site Inventory

The site inventory collects information on various site conditions and neighborhood attributes to help designers identify potential opportunities and limitations for structural SCMs, particularly GSI practices that require specific site conditions to be effective. The site inventory should take into account site conditions, as well as the neighborhood context.

Site Conditions

Existing and historic site conditions can influence stormwater management decisions. Assessment of the physical attributes and natural systems present on a site ensures that the selected stormwater management approach minimizes impacts to sensitive areas, whether natural or potentially contaminated. It also helps to identify any opportunities to take advantage of existing natural drainage patterns, reduce or remove unnecessary impervious areas, and employ the many added benefits that are achieved with properly designed sites and the implementation of green stormwater infrastructure. In Detroit, the prior use of the site may not be readily apparent. For example, currently vegetated sites may have demolition debris or former foundations below the surface that could affect infiltration. Understanding the historical use of a site may also help determine potential areas of contamination and influence stormwater management decisions. For example, if a site contains capped contaminated areas, the stormwater management opportunities on the site might call for shallow SCMs and if the site inventory reveals larger contaminated areas, the site design would likely need to avoid using those areas for stormwater management. Site condition attributes such as utilities, soils, existing structures, and climate, will also assist with future vegetation considerations for SCM selection and design. Table 3-1 identifies the most common site inventory attributes and associated tasks and information sources for compiling the inventory. However, it is important to examine the specific applicable regulatory requirements and project goals to identify additional site inventory attributes.

Table 3-1. Site Condition Attributes, Tasks, and Data Sources for Site Inventory

Site Conditions		
Attribute	**Task**	**Sources**
Topography	Identify where high points and low points are located on the site along with any steep slopes.	Topographic survey, digital elevation data.
Structures and Paved Areas	Identify the locations of infrastructure and paved areas including driveways, sidewalks, and compacted gravel surfaces. Note building materials and proximity to other buildings, locations of visibility windows, entrances and signage where views will need to be maintained. Identify emergency routes and site access/egress.	Aerial imagery, topographic survey, field inventory.
Utilities	Identify locations and sizes of existing sewer pipes, water lines, gas lines, electric and communication utilities, and all associated infrastructure such as manholes and catch basins. Identify locations of shallow utilities, any aging underground infrastructure and overhead wires that may impacted tree selection. Make note of utility poles and hydrants that will require access and visibility.	Local utility companies, as-built or record drawings, Michigan Underground Utility Safety Notification System (MISS DIG), topographic survey, field verification when necessary.
Soils	Evaluate existing soil conditions onsite including, permeability, depth to groundwater or impenetrable layers, locations of sand seams, presence of perched or seasonally high groundwater, and capped contaminated soils. Evaluate existing soils for nutrients, pH, textural characteristics, and hydrologic soil groups.	USDA Soil Surveys, geotechnical investigation reports, infiltration testing reports.
Previous Land Uses	Researching previous land uses can provide valuable information regarding what can be expected below ground. Large foundations or demolition debris may have been left, or past land uses may indicate a potential for contaminated or overly compacted soils.	Tax records, Sanborn Fire Insurance Maps, title records & plat maps, historic aerial photography, environmental surveys or investigations.
Waterbodies, Wetlands & Floodplains	Identify locations and buffers of onsite and nearby water features. Offsite water features may impact the site in the form of floodplains, regulation of wetlands, or distance to a navigable waterway, and should be noted where applicable.	Topographic survey, FEMA Flood Maps, wetland areas (e.g., MDEQ Wetlands Map Viewer), field verification when necessary.
Hydrology	Identify onsite drainage patterns, drainage pathways and discharge points off site.	Topographic survey, aerial photography, field verification
Existing Vegetation	Make note of nearby existing vegetation including maturity, health and whether or not it is native, exotic or invasive. Identify vegetation to be protected or removed.	Aerial imagery, topographic survey, field inventory.
Climate	Identify wind patterns, average minimum and maximum temperatures, sun/shade relationships, and areas of increased temperature (i.e. places surrounded by dark pavement or areas affected by bright building reflections).	Sun/shade studies, solar radiation maps, seasonal and monthly climate data, field verification.

Neighborhood Context

Understanding how visible SCMs will fit into the existing character of their surroundings is just as important as the performance of the system itself. Installations that are perceived by the client or local residents as being attractive and an amenity to the neighborhood are much more likely to be cared for and replicated. Table 3-2 highlights site inventory attributes related to the neighborhood context.

Table 3-2. Neighborhood Context Attributes, Tasks, and Data Sources for Site Inventory

Neighborhood Context		
Attribute	**Task**	**Sources**
Neighborhood Character	Make note of the current visual character of the surrounding neighborhood, including design styles, common elements and overall condition.	Aerial photography, panoramic street views, field verification, conversations with community members.
Safety	Identify sight lines that need to be preserved, both into and out of the site, signs of vandalism or neglect, and any other safety concerns pertinent to the area.	Aerial photography, panoramic street views, field verification, neighborhood crime statistics, conversations with community members.
Circulation	Identify existing circulation routes, patterns and intersections (pedestrian and vehicular), including informal ones that may need to be incorporated into the site; identify where barriers may be needed to prevent access.	Aerial photography, panoramic street views field verification, conversations with community members.

3.3.2 Site Analysis

Site analysis is the final phase of the site assessment. This involves identifying where opportunities and constraints for stormwater management exist. Displaying this information in map form assists the designer in making preliminary decisions regarding placement of programmatic elements on the site, ways to take advantage of existing site features to manage stormwater runoff, and placement of SCMs needed to achieve project requirements and goals.

3.4 Begin Design By Reducing Impervious Surfaces and Stormwater Management Needs

With a complete site assessment, the next step in the integrated site design approach is to initiate the design process by identifying opportunities to reduce impervious surfaces and the need for new structural SCMs.

One way to achieve this is by using site design principles for stormwater management that employ a combination of GSI practices and planning techniques to reproduce natural hydrologic conditions, often collectively referred to as low impact development (LID) techniques. By designing to minimizing impervious surfaces and infiltrating, evaporating, and storing stormwater runoff onsite, these principles aid in maintaining a more hydrologically functional landscape, even in dense urban settings. More detail on LID concepts for southeast Michigan is available in the Southeast Michigan Council of Governments (SEMCOG) *Low Impact Development Manual (LID) for Michigan: A Design Guide for Implementers and Reviewers* (2008), referenced in Chapter 1 of this Design Manual. A summary of key site design principles that will help reduce impervious surfaces and reduce the need for constructed SCMs is provided below.

Disconnected impervious sidewalk draining onto lawn
Source:

1. Preserve & Protect Existing Sensitive Natural Features

- Provide buffers to streams and wetlands
- Incorporate native or existing, non-invasive vegetation into the design
- Preserve existing healthy trees whenever possible
- Protect existing soils that are already providing infiltration

2. Minimize the Need for Constructed Stormwater Management Practices

- Cluster development to reduce impervious surfaces
- Remove impervious surface whenever possible
- Disconnect impervious surfaces

Decorative cistern
Source:

3. Integrate Natural Surface Hydrology & Soil Conditions into the Design

- Look for natural surface drainage patterns
- Locate stormwater management practices in low spots
- Locate infiltrating practices on soils with high infiltration rates
- Utilize areas with poor quality or contaminated soils for proposed buildings and parking areas
- Minimize soil disturbance to preserve infiltration capacity of natural soils where disconnected impervious surfaces can discharge

4. Treat Stormwater as an Amenity

- Look for ways to reuse water for non-potable uses
- Incorporate GSI practices that provide social, ecological and economic benefits

5. Treat Stormwater at its Source

- Detain and retain runoff as close to the source as possible
- Use distributed practices

Soil Restoration

A technique used to enhance and restore soils by physical treatment and/or by mixing with additives – such as compost – in areas where soil has been compacted. Soil restoration increases the water retention capacity of soil, reduces erosion, improves soil structure, immobilizes and degrades pollutants (depending on soil media makeup), supplies nutrients to plants, and provides organic matter. Soil restoration is also used to reestablish the soil's long-term capacity for infiltration and to enhance the vitality of the soil as it hosts all manner of microbes and plant root systems in complex, symbiotic relationships. Soil restoration techniques from the SEMCOG *Low Impact Development Manual for Michigan* include tilling the soil (also referred to as scarification, ripping or subsoiling) and applying soil media for amendment.

Design considerations that limit or reduce the amount of impervious cover and increase the amount of disconnected impervious surfaces are the preferred methods for managing stormwater generated on site. These methods should be maximized first whenever possible. However, in urban environments space constraints can often make achieving this quite difficult, and additional design solutions such as on-site soil restoration and constructed stormwater management practices will be necessary.

In addition to looking at the site for ways to reduce or remove impervious surfaces, landscape design principles applied early in the design process can assist in bringing multiple benefits to the site that will ensure stormwater management effectiveness embraced by community members. Landscape design aims to combine art and science to create functional and aesthetic designs that consider the needs of the users as well as the needs of the site. The following principles should be considered when designing spaces that will interact with people, support plant material, and integrate ecological sustainability.

1. Aesthetics

- Create stormwater management practices that look cared for and intentional by creating defined perimeters and designing maintainable spaces
- Design elements that fit into or enhance the existing neighborhood character
- Plant in recognizable patterns, use plant material the public is familiar with, and incorporate vegetation that provides interest in all seasons
- Design elements that create visual interest

Planter boxes included in street design in urban corridor
Source:

2. Provide Social & Cultural Benefits

- Protect and provide trees whenever possible to create shade, reduce surface temperatures, and clean the air

- Include natural features and open spaces which have been shown to reduce stress levels, increase levels of physical activity, and improve reported well-being

3. Maximize Long-term Economic Benefits

- Reduce building energy needs through selective vegetation placement (deciduous trees on the south and west sides, evergreens on northwest side)
- Trees and green stormwater infrastructure have been shown to increase property values and increase the amount people are willing to pay to live in places with these features

4. Safety

- Identify and maintain desired sight lines
- Do not design elements that can create hiding places or become hazards in perceived high crime areas or where children will be present
- Special safety considerations are needed when planting near roads, intersections and circulation routes. Maintain applicable clear zones and keep vegetation below 3 feet in height (at maturity).

5. Enhance Ecological Value

- Eliminate invasive species
- Increase local biodiversity by providing a variety of plant species, especially natives
- Include trees, shrubs and herbaceous vegetation to provide benefits for wildlife like food, habitat and shelter. Design green spaces to provide habitat connectivity when possible.

6. Planting Design that Plans for Maintenance

- Know what maintenance will be available for the project, including how much and the experience of the crews with GSI, before beginning design
- Design for fewer weeds by reducing the area of bare ground by increasing planting densities to achieve 90 percent cover within two growing seasons
- Plant in ways that reduces the amount of knowledge necessary to perform required maintenance tasks. Options include:
  - Planting with a restricted palette (only 1-3 species)
  - Planting in obvious patterns (masses, rows)
  - Select plant material that can be mown annually or semi-annually

Plantings in obvious patterns in a bioretention garden
Source:

GSI planting event with local students
Source:

Trees providing cover along Riverwalk in Detroit
Source:

Low plantings near high use circulation routes
Source:

3.5 Review Options for Site SCMs

After identifying opportunities to reduce impervious surfaces and the need for stormwater management through site design, the next step in the integrated site design approach is to review both structural and non-structural SCM options to effectively manage stormwater from the impervious surfaces on the remainder of the site. The process of selecting the most effective and appropriate structural SCMs for a site can be challenging, especially when faced with space constraints and tough site conditions. Testing various configurations of required site elements and site layouts allows the designer to meet the goals and objectives for a project in an efficient and cost-effective manner. Non-structural SCMs can assist with pollution prevention to help minimize contamination of stormwater runoff from the site.

3.5.1 Structural SCM Technical Review

There are many options structural SCMs to consider for a site. Selecting the right SCM, or suite of SCMs, requires an understanding of the different SCM functions, costs, added benefits, and interactions between other site functions. This Design Manual provides detailed information on a variety of SCMs in Chapters 5-14. These SCM specific chapters go into detail on the site characteristics and technical requirements needed to effectively implement each SCM. Reviewing the information in these SCM specific chapters should be a first step in considering which SCMs might be appropriate for a site prior to initiating a conceptual design.

Three examples of green stormwater infrastructure practices in commercial district with similar benefits but different designs and costs. From left to right: tree based suspended pavement (Philadelphia PA), curb extension bioretention (Portland OR), and planter box (Lansing, MI).
Source:

3.5.2 Non-Structural Source Control SCM Opportunity Review

Non-structural source control SCMs are intended to prevent or reduce the generation of pollutants in stormwater using practices that focus on facility operations and procedures. These SCMs are commonly used at commercial and industrial sites where potential pollutant sources may be exposed to stormwater runoff. Non-structural source control SCMs to consider during site design are described below.

Pollution Prevention

The best way to prevent stormwater pollution is to minimize the use of pollutants in commercial and industrial activities. This could include reducing the use of a product, or substituting a less toxic product to use. Another effective pollution prevention practice is to move materials and activities indoors, where they will not be exposed to stormwater runoff.

Educational Signage

Educating Detroiters about the connection of the street to the sewer system to local waterways is a key step in keeping pollutants and solids out of the sewer system. One way to do this is through storm drain stenciling with a no dumping message that will resonate with Detroiters about where pollutants and trash could end up once it enters the sewer system. Post signage near waterways that prohibit illegal dumping. Consider educational signage near structural SCMs that explain the function of the measure and Dos and Don’ts that will affect performance of the SCM.

Storm drain stencil label
Source: EPA

Good Housekeeping

Good housekeeping practices offer a practical and cost-effective way to maintain a clean and orderly facility to prevent potential pollution sources from coming into contact with stormwater. Good housekeeping practices also help to enhance safety and improve the overall work environment. Good housekeeping practices include:

- Maintaining a clean workplace through frequent sweeping
- Regular collection and disposal of garbage and waste material
- Cover and maintain dumpsters and waste receptacles
- Identify and label all containers, including with information from Material Safety Data Sheets (MSDS)
- Train employees on good housekeeping practices and publicize good housekeeping practices using posters or signs

Trash Storage Areas

Trash storage areas should be on an impervious surface designed to prevent run-on from adjoining areas. Trash containers or dumpsters should have lids to prevent rainfall or snowfall intrusion. A roof or cover could be considered for high use trash areas.

Outdoor Material Handling and Storage Areas

Where practical, conduct operations indoors. Store bulk solid materials such as raw materials, sand, gravel, topsoil, compost, concrete, packing materials, metal products and other materials covered in accordance with the Detroit Bulk Solid Storage Ordinance and protected from stormwater. When practical, store materials on impermeable surfaces. Store hazardous materials according to federal, state, and local hazardous materials requirements and use secondary containment structures. Inspect temporary covers, such as tarps, frequently and repair/replace when torn or materials are exposed. If transporting materials to and from the storage area, cover the materials during conveyance and to reduce environmental dust and wash associated with fine particulate matter.

DWSD maintains a 24-Hour Emergency Maintenance Hotline: 313-267-7401 for reporting any water and sewer emergency issues including spills, overflows or sewer backups.

Covering Loading/Unloading Dock Areas

Provide overhead cover when appropriate to prevent precipitation coming into contact with materials. Isolate drainage in the loading dock area through the use of paved berms and/or grade breaks to prevent adjacent runoff from entering the loading area and to prevent liquid spills from discharging from the loading area.

Covered material storage area
Source:

Vehicle/Equipment Maintenance

Conduct maintenance activities indoors when possible. Perform regular inspection and preventative maintenance of vehicles/equipment to ensure proper operation and to check for leaks. Use drip pans to collect leaks and spills from vehicles and equipment, and empty drip pans regularly. Drain oil and fuel filters and dispose of them into appropriately closed and properly labeled containers. Conduct maintenance activities away from storm drains.

Clearly labeled spill kit
Source:

Vehicle/Equipment Fueling

Keep spill kits next to fueling areas with clear signage. Clean up spills with dry methods (absorbents) and use damp cloths on gas pumps and damp mops on paved surfaces. Never use a hose to “wash down” a fuel spill. If possible, fuel-dispensing areas should be paved with concrete and covered. Regularly inspect fueling areas to check for spills, leaks, corrosion, or other damage.

Vehicle/Equipment Wash Areas

Where practical, keep vehicle wash areas self-contained with discharge to a properly permitted connection to a sanitary sewer.

Spill Prevention and Response Procedures

Spills and leaks, together, are the largest source of commercial and industrial stormwater pollution. Train employees on spill prevention practices. Keep spill containment and cleanup kits on-site and readily accessible, and clean up all spills immediately upon discovery. Do not flush any spill or cleanup materials into a storm drain – use dry sweep methods and dispose in appropriate containers.

Erosion and Sediment Controls

Limit erosion on areas of the site that, due to topography, land disturbing activities, soils, cover, materials, or other factors, are likely to experience erosion. In general, erosion control measures, which prevent soil or sediment from becoming mobilized, should be used as the primary line of defense, while sediment control measures, which trap, infiltrate, or settle out mobilized sediments, should be used to back-up the erosion control measures. For instance, erosion control measures, include grading, seeding, mulching, and sodding that prevent soil from becoming dislodged, should be considered first. Where sediment may be dislodged and potentially mobilized in stormwater runoff, sediment control measures that trap eroded sediment, such as silt fences, sediment ponds, and stabilized entrances should be considered.

Snow and Ice Management

Designate snow storage areas in locations that direct runoff to GSI practices for treatment, where practicable. Deicing chemicals can have a severe impact on plants and vegetation in GSI practices – carefully consider the need of deicing chemical in areas that drain to vegetation. Do not plow snow directly into streams.

Use signage for deicing near GSI practices
Source:

Employee Training

Train employees who work in areas with pollutants that could be exposed to stormwater on how to properly handle pollutants, respond to spills, and report water quality problems.

3.6 Prepare Conceptual Site Design

With an understanding of the project goals, applicable performance standards, site assessment, and stormwater management opportunities, and a suite of possible SCMs based on the technical review, creating a conceptual design for the site is now feasible. An iterative conceptual design process is recommended, allowing the site design process to test the potential use of several selected SCMs to find those that are most optimal for the project. The steps in the iterative conceptual design process, shown in Figure 3-2, can can save time and money by identifying any conflicts or performance issues early in the process before other site design elements are finalized. Using the applicable site and landscape design principles, the steps can be implemented, assessed, and adjusted until an appropriate solution is reached.

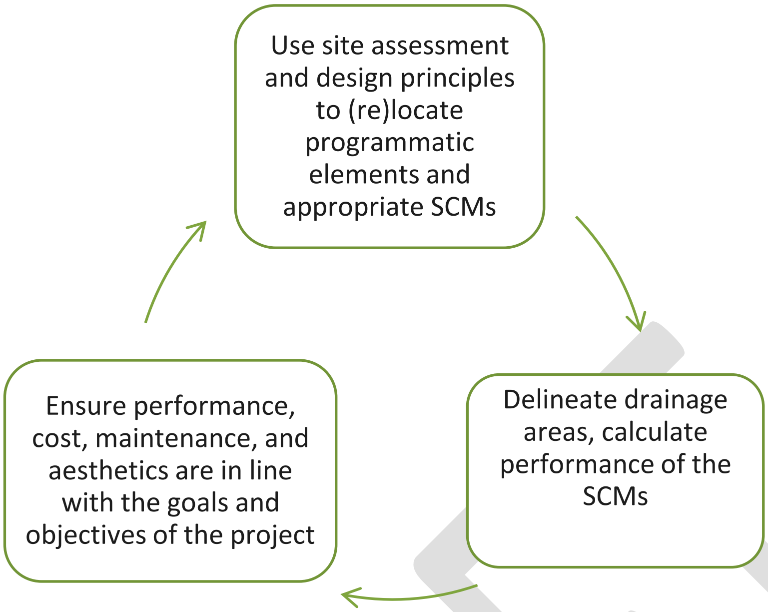


Figure 3-2. Iterative Conceptual Design Process

3.7 Begin Final Design and Assemble Post-Construction Stormwater Management Plan Submittal

Once the concept design has satisfactorily met all project goals and objectives, the design can be finalized, and the Post-Construction Stormwater Management Plan can be prepared. **[Insert new text with input from DWSD on final design phases and review considerations]**

Once the final design is complete, the last step in the integrated site design process is to assemble the documentation required for the PCSWMP. Chapter 2 of this Design Manual identified the required components of the PCSWMP. To ensure complete PCSWMP submittal, use the PCSWMP application form and checklist of required documentation provided in Appendix A.

3.8 References

Center for Neighborhood Technology (2010). The Value of Green Infrastructure: A Guide to Recognizing Its Economic, Environmental and Social Benefits.

LaGro Jr, J.A. (2008). Site analysis: A contextual approach to sustainable land planning and site design, John Wiley & Sons.

SEMCOG (2008). Low Impact Development Manual for Michigan: A Design Guide for Implementors and Reviewers.

Tzoulas, Konstantinos, et al. (2007). Promoting ecosystem and human health in urban areas using Green Infrastructure: A literature review. *Landscape and urban planning*, 81(3), 167-178.

4. Hydrologic Procedures

This chapter provides climatological information, temporal rainfall distributions and acceptable methods to use for calculating stormwater runoff. The hydrologic procedures provided in this chapter are common procedures that are routinely used. All the procedures can be accomplished with software. The basic procedures result in computing a peak flow rate or a volume. For many design situations, a complete hydrograph is needed.

Contents

4.1 Hydrology

4.1.1 Precipitation Frequency Data

The rainfall data presented in Table 4-1 is from *NOAA Atlas 14 Precipitation-Frequency Atlas of the United States, Volume 8 Version 2: Midwestern States, including Michigan*, based on the weather station at the Detroit Metropolitan Airport. The same information is shown graphically in the Intensity-Duration-Frequency (IDF) curves in Figure 4-1.

Table 4-1 Precipitation Depth (in) for Recurrence Interval (years)

Duration	1 year	2 year	5 year	10 year	25 year	50 year	100 year
5 min	0.31	0.37	0.46	0.54	0.66	0.74	0.83
10 min	0.46	0.54	0.68	0.80	0.96	1.09	1.22
15 min	0.56	0.66	0.83	0.97	1.17	1.33	1.49
30 min	0.76	0.90	1.13	1.33	1.61	1.83	2.05
1 hour	0.97	1.15	1.45	1.70	2.07	2.36	2.66
2 hour	1.18	1.40	1.76	2.08	2.53	2.89	3.26
3 hour	1.31	1.55	1.95	2.29	2.80	3.20	3.63
6 hour	1.55	1.80	2.24	2.64	3.21	3.69	4.19
12 hour	1.80	2.06	2.52	2.94	3.57	4.09	4.65
24 hour	2.06	2.35	2.85	3.31	3.98	4.55	5.15
2 day	2.35	2.69	3.27	3.78	4.52	5.12	5.75
3 day	2.58	2.93	3.54	4.06	4.82	5.44	6.08
4 day	2.78	3.14	3.76	4.30	5.07	5.70	6.34
7 day	3.29	3.69	4.36	4.94	5.77	6.43	7.11

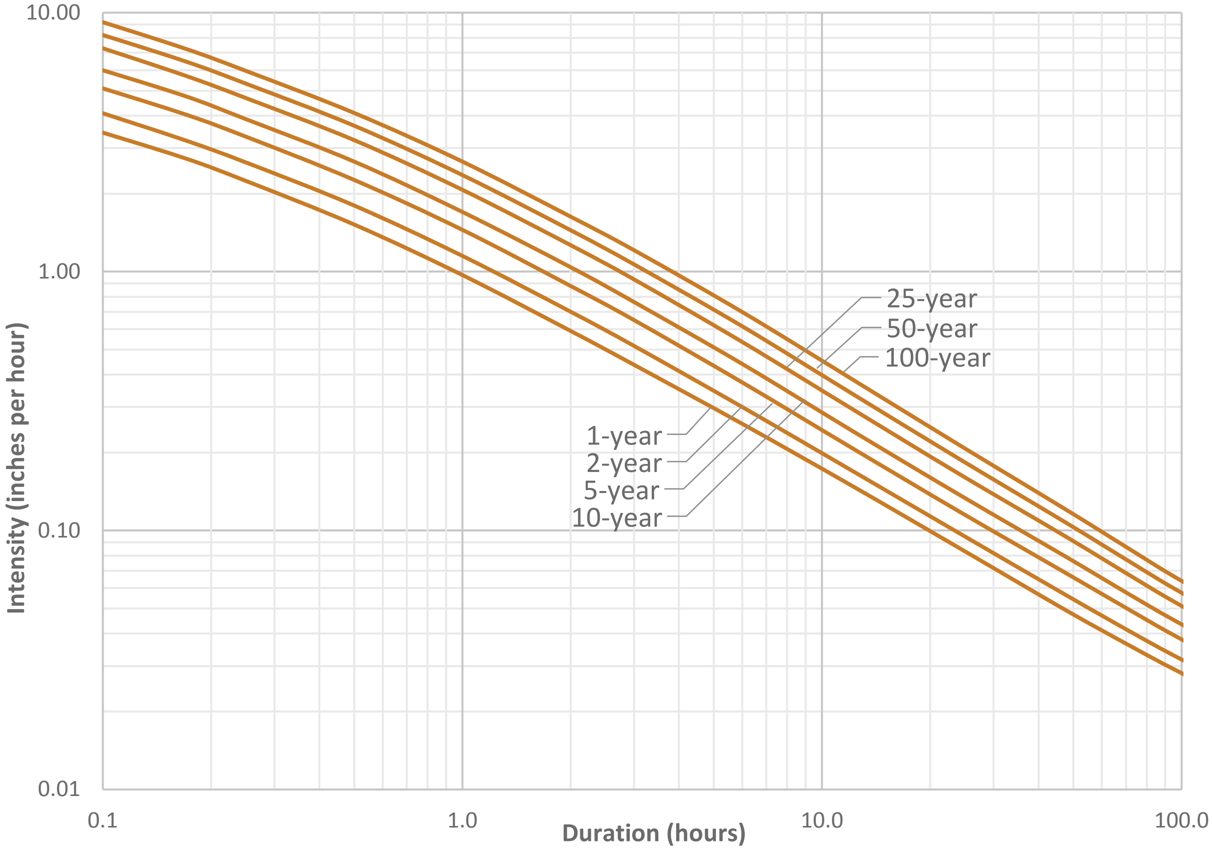


Figure 4-1 Intensity Duration Frequency (IDF) Curve

An equation form of the intensity-duration-frequency curve is also available.

$$i = \frac{38.4164T^{0.2082}}{(12.3258 + D)^{0.8405}} \tag{4.1}$$

where i = design rainfall intensity, in/hr.
T = return period, yr.
D = duration, min

The coefficients for the equation form of the IDF curve were developed from a regression analysis for durations from 5 minutes to 24 hours.

4.1.2 Small Non-Exceedance Storms

Small storms are responsible for most annual urban runoff and most pollutant wash-off from urban surfaces. Large storms contribute significant per event runoff volumes; however, they tend to occur infrequently. A non-exceedance rainfall analysis is presented in Figure 4-2 based on hourly rainfall data from the Detroit Metropolitan

Airport from 1958 to 2013 for runoff producing events. For regulatory purposes, runoff producing events are assumed to be rainfall events larger than 0.10 inches.

The 90 percent non-exceedance precipitation event is 1.0-in. This means that 90 percent of all events during an average year are less than or equal to 1.0-in. of rainfall.

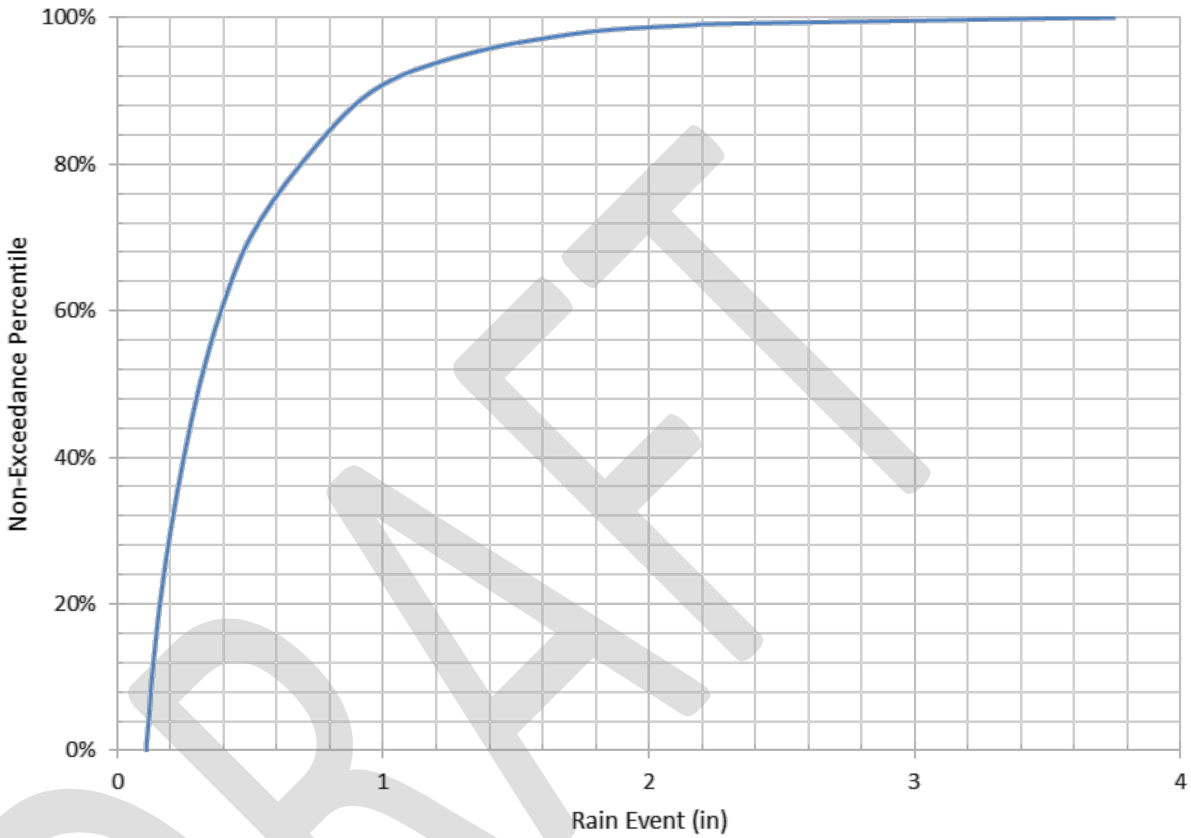


Figure 4-2 Non-Exceedance Rainfall

4.1.3 Continuous Rainfall Data

An analysis using continuous rainfall data may be used for some calculations. Continuous rainfall data may be helpful for designing green infrastructure and water harvesting practices but should not be used for sizing conveyance systems or large storage basins. When performing a continuous rainfall analysis, a minimum of 10 years of continuous rainfall data must be used. Rainfall data shall be hourly or smaller increments, and shall be from a National Weather Service station such as the Detroit Metropolitan Airport.

4.1.4 Rainfall Distribution

When a runoff hydrograph is required for design calculations a temporal rainfall distribution is needed. Historically a NRCS Type II rainfall distribution has been used.

NRCS is replacing the use of the legacy rainfall distributions (Type I, IA, II, and III) with rainfall distributions based on NOAA Atlas 14 precipitation-frequency data. These rainfall distributions are based on the 5-minute through 24-hour rainfall depths for a specific return period. For the Detroit area a NRCS Midwest-Southeast (MSE) Type 3 rainfall distribution shall be used. This distribution is intended to be used for design purposes.

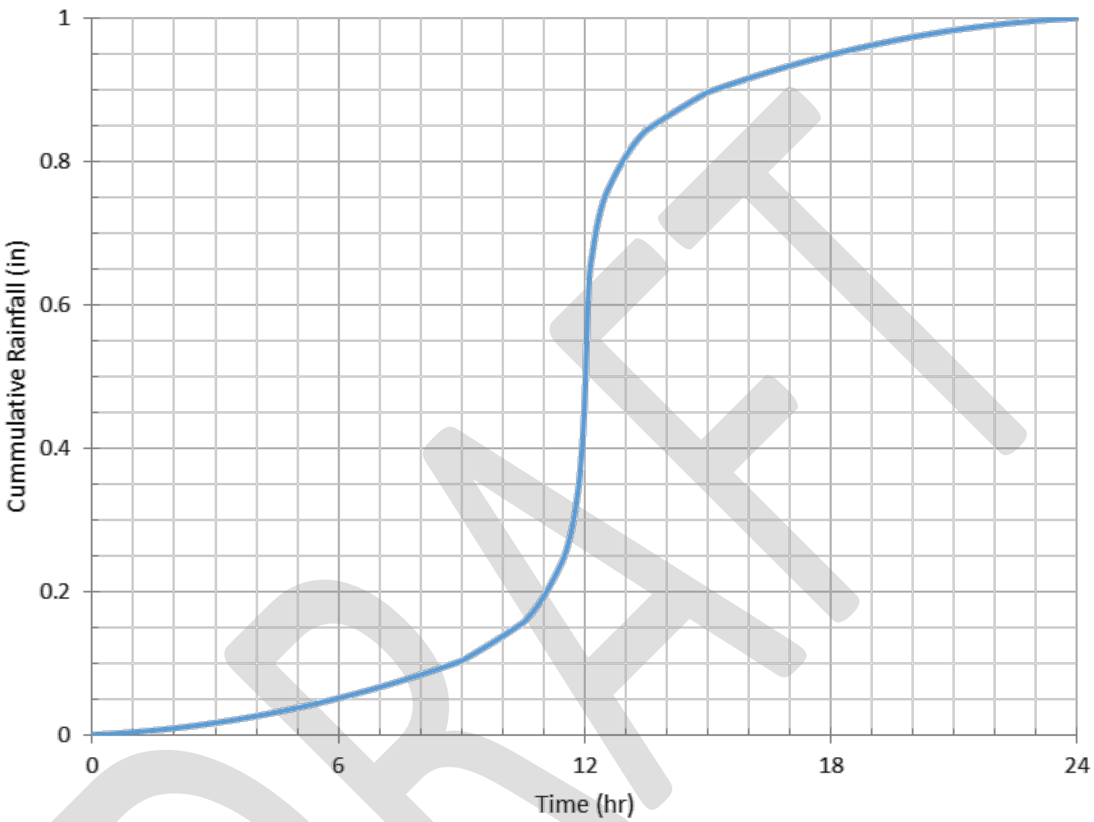


Figure 4-3 MSE3 Rainfall Distribution

Table 4-2 MSE3 Rainfall Distribution

Time (hr.)	Rainfall (in)	Time (hr.)	Rainfall (in)	Time (hr.)	Rainfall (in)	Time (hr.)	Rainfall (in)	Time (hr.)	Rainfall (in)	Time (hr.)	Rainfall (in)
0.1	0.00027	4.1	0.02714	8.1	0.08556	12.1	0.62755	16.1	0.91811	20.1	0.97496
0.2	0.00056	4.2	0.02821	8.2	0.08742	12.2	0.67555	16.2	0.91992	20.2	0.97597
0.3	0.00086	4.3	0.02931	8.3	0.08931	12.3	0.70938	16.3	0.92170	20.3	0.97697
0.4	0.00119	4.4	0.03042	8.4	0.09121	12.4	0.73370	16.4	0.92347	20.4	0.97795
0.5	0.00153	4.5	0.03156	8.5	0.09314	12.5	0.75200	16.5	0.92522	20.5	0.97891
0.6	0.00190	4.6	0.03271	8.6	0.09508	12.6	0.76457	16.6	0.92694	20.6	0.97984
0.7	0.00229	4.7	0.03389	8.7	0.09704	12.7	0.77638	16.7	0.92865	20.7	0.98076
0.8	0.00269	4.8	0.03508	8.8	0.09903	12.8	0.78744	16.8	0.93034	20.8	0.98166
0.9	0.00312	4.9	0.03630	8.9	0.10103	12.9	0.79774	16.9	0.93201	20.9	0.98254
1.0	0.00356	5.0	0.03753	9.0	0.10305	13.0	0.80728	17.0	0.93365	21.0	0.98340
1.1	0.00403	5.1	0.03878	9.1	0.10628	13.1	0.81606	17.1	0.93528	21.1	0.98424
1.2	0.00451	5.2	0.04006	9.2	0.10956	13.2	0.82409	17.2	0.93689	21.2	0.98506
1.3	0.00501	5.3	0.04135	9.3	0.11289	13.3	0.83137	17.3	0.93848	21.3	0.98586
1.4	0.00554	5.4	0.04266	9.4	0.11626	13.4	0.83788	17.4	0.94005	21.4	0.98664
1.5	0.00608	5.5	0.04399	9.5	0.11967	13.5	0.84364	17.5	0.94160	21.5	0.98740
1.6	0.00665	5.6	0.04535	9.6	0.12314	13.6	0.84752	17.6	0.94313	21.6	0.98814
1.7	0.00723	5.7	0.04672	9.7	0.12664	13.7	0.85134	17.7	0.94464	21.7	0.98886
1.8	0.00783	5.8	0.04811	9.8	0.13020	13.8	0.85513	17.8	0.94613	21.8	0.98956
1.9	0.00845	5.9	0.04952	9.9	0.13380	13.9	0.85887	17.9	0.94760	21.9	0.99024
2.0	0.00910	6.0	0.05095	10.0	0.13744	14.0	0.86256	18.0	0.94905	22.0	0.99090
2.1	0.00976	6.1	0.05240	10.1	0.14113	14.1	0.86620	18.1	0.95048	22.1	0.99155
2.2	0.01044	6.2	0.05387	10.2	0.14487	14.2	0.86980	18.2	0.95189	22.2	0.99217
2.3	0.01114	6.3	0.05536	10.3	0.14866	14.3	0.87336	18.3	0.95328	22.3	0.99277
2.4	0.01186	6.4	0.05687	10.4	0.15248	14.4	0.87686	18.4	0.95465	22.4	0.99335
2.5	0.01260	6.5	0.05840	10.5	0.15636	14.5	0.88033	18.5	0.95601	22.5	0.99392
2.6	0.01336	6.6	0.05995	10.6	0.16212	14.6	0.88374	18.6	0.95734	22.6	0.99446
2.7	0.01414	6.7	0.06152	10.7	0.16863	14.7	0.88711	18.7	0.95865	22.7	0.99499
2.8	0.01494	6.8	0.06311	10.8	0.17591	14.8	0.89044	18.8	0.95994	22.8	0.99549
2.9	0.01576	6.9	0.06472	10.9	0.18394	14.9	0.89372	18.9	0.96122	22.9	0.99597
3.0	0.01660	7.0	0.06635	11.0	0.19272	15.0	0.89695	19.0	0.96247	23.0	0.99644
3.1	0.01746	7.1	0.06799	11.1	0.20226	15.1	0.89897	19.1	0.96370	23.1	0.99688
3.2	0.01834	7.2	0.06966	11.2	0.21256	15.2	0.90097	19.2	0.96492	23.2	0.99731
3.3	0.01924	7.3	0.07135	11.3	0.22362	15.3	0.90296	19.3	0.96611	23.3	0.99771
3.4	0.02016	7.4	0.07306	11.4	0.23543	15.4	0.90492	19.4	0.96729	23.4	0.99810
3.5	0.02109	7.5	0.07478	11.5	0.24800	15.5	0.90686	19.5	0.96844	23.5	0.99847
3.6	0.02205	7.6	0.07653	11.6	0.26630	15.6	0.90879	19.6	0.96958	23.6	0.99881
3.7	0.02303	7.7	0.07830	11.7	0.29062	15.7	0.91069	19.7	0.97069	23.7	0.99914
3.8	0.02403	7.8	0.08008	11.8	0.32445	15.8	0.91258	19.8	0.97179	23.8	0.99945
3.9	0.02504	7.9	0.08189	11.9	0.37245	15.9	0.91444	19.9	0.97286	23.9	0.99973
4.0	0.02608	8.0	0.08371	12.0	0.46289	16.0	0.91629	20.0	0.97392	24.0	1.00000

4.1.5 Rainfall Inter-Event Durations

Table 4-3 presents information on the number of dry days preceding a rain event based on 53 years of rainfall records at the Detroit Metropolitan Airport. An example for interpreting the data in Table 4-3 is as follows: over 53 years (1959 to 2013) there have been 28 times when at least 7 days of dry weather before a rain event during the month of June. On average then we have a period of no rain for at least 7 days in June about once every two years (53 years divided by 28 occurrences). Similarly, during summer months, June thru August, there have been 35 times when there was at least 10 days of dry weather and 6 times when there was at least 14 days of dry weather.

Table 4-3 Number of Dry Days Preceding a Rain Event[1]

Dry Days	Number of Occurrences Over 53 years (1959 to 2013)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Jun-Aug
> 3 days	73	71	85	98	110	99	127	117	102	107	93	97	1179	343
> 5 days	34	26	34	46	53	49	68	62	56	63	51	40	582	179
> 7 days	14	15	16	19	21	28	29	30	33	34	28	15	282	87
> 10 days	3	7	3	5	10	9	14	12	16	14	11	6	110	35
> 14 days	0	2	1	2	1	2	2	2	6	2	0	1	21	6
> 21 days	0	0	0	0	0	0	1	0	1	1	0	0	3	1

1. Only rainfall events greater than 0.10 inches of rainfall were considered for this analysis.

The information in Table 4-3 is useful when planning for periods of drought for issues such as water harvesting, vegetation management or constructed stormwater wetlands. The determination of the selected drought duration is the designer's responsibility. Different stormwater management designs have a different tolerance for the drought duration. For example, a rainwater harvesting system used as supplemental irrigation may be planned for a short 3-day drought duration to keep the tank size small whereas a constructed stormwater wetland may need to plan for 2 or more weeks of dry weather if supplemental water is not available.

4.1.6 Evaporation

Monthly pan evaporation rates are provided in Table 4-4 (NOAA, 1982). Pan evaporation is literally the measured rate of evaporation from a pan. Evaporation from a natural water body is usually at a lower rate than evaporation from a pan. Summer evaporation rates may be estimated as 0.75 times the local pan evaporation rate. The rate of evaporation varies based on direct sunlight, wind, air pressure, temperature, and humidity.

Table 4-4 Monthly Evaporation

Month	Dearborn Class A Pan Evaporation Monthly (inches)	Detroit Metro Airport Estimated Pan Evaporation Monthly (inches)
January		0.87
February		1.21
March		2.16
April	3.88	3.69
May	5.86	5.43
June	6.91	6.54
July	7.35	6.85
August	6.18	5.9
September	3.14	4.17
October	2.99	3.07
November		1.62
December		1.00

4.1.7 Monthly Precipitation

Table 4-5 provides a summary of monthly rainfall based on the Detroit Metropolitan Airport. Twenty-five (25) years of historical monthly precipitation is provided along with a statistical summary for the average, high and low. Data is based on daily rainfall values.

Table 4-5 Monthly Precipitation (inches)

Statistical	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average	**2.25**	**2.09**	**2.37**	**3.16**	**3.61**	**3.39**	**3.45**	**3.22**	**3.05**	**2.36**	**2.49**	**2.28**	**33.71**
High	3.92	3.90	4.86	5.61	8.46	6.05	7.66	6.61	6.71	6.76	6.00	4.07	47.70
Low	0.42	0.63	0.74	0.69	1.18	0.94	1.16	0.27	0.62	0.13	0.62	0.78	27.11
Days > 0.1 in [a]	6	5	6	7	7	6	6	6	6	5	5	5	72
Historical	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Annual**
1993	3.92	1.27	2.12	3.32	1.24	6.05	2.17	1.60	4.26	2.21	1.69	0.78	30.63
1994	2.79	1.38	2.29	4.04	1.18	3.97	3.20	3.30	2.38	1.35	2.74	2.39	31.01
1995	2.47	0.89	1.73	3.44	3.55	1.55	3.40	3.71	0.62	3.53	3.08	0.85	28.82
1996	1.85	1.76	1.56	3.39	2.82	2.37	2.64	0.43	4.42	1.59	1.99	2.57	27.39
1997	2.35	3.90	3.22	1.56	5.23	3.17	2.68	3.22	3.41	1.91	0.94	1.61	33.20
1998	2.60	3.56	3.62	3.86	2.46	2.69	5.72	4.19	1.50	1.41	1.36	1.16	34.13
1999	3.00	1.98	1.12	5.13	2.20	5.46	3.62	1.31	3.11	1.56	1.49	2.22	32.20
2000	1.29	0.84	1.55	4.35	5.11	4.90	5.40	4.63	6.71	3.05	1.69	2.63	42.15
2001	0.69	2.88	0.93	3.20	3.70	3.40	1.16	2.87	4.28	6.76	2.35	2.23	34.45
2002	3.36	1.91	2.12	4.48	3.76	1.07	3.50	3.32	1.95	1.15	2.72	1.16	30.50
2003	0.42	0.66	1.46	2.07	4.73	2.50	2.59	4.36	4.27	2.74	2.97	2.62	31.39
2004	1.43	0.63	3.29	0.69	8.46	2.86	2.85	4.51	0.65	2.08	3.21	2.91	33.57
2005	3.40	3.02	0.74	1.66	1.85	1.95	5.38	1.33	1.63	0.13	4.70	2.52	28.31
2006	3.24	2.71	3.21	2.71	4.60	3.95	4.38	2.05	1.73	4.11	2.90	3.65	39.24
2007	3.02	0.82	3.09	2.68	2.56	3.10	2.10	6.61	1.44	2.00	1.77	3.48	32.67
2008	2.13	3.61	3.17	0.96	2.03	4.05	3.24	0.27	5.99	1.15	3.31	4.07	33.98
2009	1.10	2.12	4.17	5.03	2.89	5.27	2.56	2.76	1.46	3.23	0.62	2.90	34.11
2010	0.76	1.90	1.07	2.26	5.31	5.42	5.96	0.59	3.32	1.07	3.34	1.28	32.28
2011	1.53	3.60	3.61	5.61	5.38	0.94	7.66	2.16	6.28	2.14	6.00	2.79	47.70
2012	3.00	1.91	2.95	2.15	1.72	1.31	3.67	2.25	2.47	2.32	0.72	2.64	27.11
2013	3.45	2.83	0.74	5.29	2.54	6.01	4.14	5.98	1.20	3.48	1.82	2.42	39.90
2014	2.92	2.82	1.49	2.57	4.87	4.00	2.43	6.32	4.71	2.36	1.67	1.41	37.57
2015	1.45	1.35	0.80	2.61	5.54	5.32	1.76	3.16	1.29	1.97	2.06	3.01	30.32
2016	1.34	2.02	4.86	2.31	2.20	1.30	1.57	5.62	6.28	2.98	2.10	2.16	34.74
2017	2.83	1.90	4.26	3.55	4.39	2.02	2.44	3.91	0.91	2.84	4.93	1.48	35.46

a. Average number of days with 0.10 in. precipitation or more

4.1.8 Climate Change

As climate change warms the atmosphere, altering the hydrologic cycle, changes to the amount, timing, form, and intensity of precipitation will continue (US Environmental Protection Agency, 2017). Southeast Michigan has seen an 11% increase in total annual precipitation from the 1961-1990 average to the 1981-2010 average and most models project this trend to continue. Extreme precipitation events have become more frequent and more intense (Great Lakes Integrated Sciences + Assessments, 2013).

At this time designing stormwater management systems and practices for estimated future climate changes is not required.

4.2 Computational Methods

There are different methods to calculate the stormwater runoff volume and peak flow rate from a given storm. Table 4-6 identifies the approved methodologies to use for calculating and routing the runoff and are discussed in further detail in the sections following the table. Other methods may be used with approval from the Department.

Table 4-6 Computational Methods

Process Description	Approved Methodologies
Stormwater conveyance system sizing, including routing flow offsite through the municipal collection system	Rational Method, or EPA SWMM
Surface runoff volume for the water quality storm	Small Storm Hydrology or Simple Method
Surface runoff generation for discrete design storms (1- to 100-yr events)	NRCS Curve Number Approach, or EPA SWMM
Detention basin volumetric sizing for large storms (10- to 100-yr events)	Modified Rational Method (1), NRCS Curve Number Approach, or EPA SWMM
Routing flow on site through SCMs and stormwater controls	Approved hydrology modeling packages that use and route the NRCS unit hydrograph, or EPA SWMM

(1) Modified Rational Method is suitable for conceptual design sizing of detention basins but not for final design calculations.

4.2.1 Time of Concentration

The time of concentration (t_c) is the time required for water to travel from the hydraulically most remote point of the basin to the point of interest. The t_c must be determined to be able to use the Rational Method to estimate peak flow for sizing storm sewer systems, or for applying unit hydrographs and NRCS curve number methods to generate and route runoff hydrographs for sizing storm sewer systems and stormwater controls.

The velocity method should be used for calculating the time of concentration. The velocity method assumes that the time of concentration is the sum of travel times for each segment along the longest flow path across the drainage area. Segments along the flow path are typically composed of sheet flow, shallow concentrated flow, open channel flow, and the residence time in storage systems. Refer to the *National Engineering Handbook Part 630 Hydrology*, Chapter 15 for a complete discussion of time of concentration (Natural Resources Conservation Service, 2010).

Sheet Flow

Sheet flow is overland flow of water in a thin continuous layer over the ground surface. Travel time for sheet flow may be computed with the following equation:

$$T_t = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}} \tag{4.2}$$

where T_t = travel time (hours)
n = Manning's roughness coefficient for sheet flow (Table 4-7)
l = sheet flow length, ft.
P_2 = 2-year 24-hour rainfall, in (Table 4-1)
S = slope of land surface, ft./ft.

The maximum sheet flow length (l) allowed is:

$$l = \frac{100\sqrt{S}}{n} \tag{4.3}$$

Table 4-7 Manning's Roughness Coefficients for Sheet Flow[1]

Surface Description	Manning 'n' Recommended Value	Manning 'n' Range
Concrete or asphalt	0.011	0.01 – 0.013
Graveled surface	0.02	0.012 – 0.03
Bare clay-loam (eroded)	0.02	0.012 – 0.033
Grass		
Short-grass prairie	0.15	0.10 – 0.20
Dense grasses[2]	0.24	0.17 – 0.30
Bermudagrass	0.41	0.30 – 0.48
Bluegrass sod	0.45	0.39 – 0.63
Woods		
Light underbrush	0.40	
Dense underbrush	0.80	
Agriculture		
Fallow (no residue)	0.05	0.006 – 0.16
Cultivated soil, residue cover ≤20%	0.06	
Cultivated soil, residue cover ≥ 20%	0.17	
Rangeland		
Natural	0.13	0.01 – 0.32
Clipped	0.10	0.02 – 0.24

1. Sheet flow generally has flow depths ≤ 0.1 ft.
2. Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue gamma grass, alfalfa, and lespedeza

Reference: (Engman, 1986) (Welle & Woodward, 1986) (Natural Resources Conservation Service, 2010)

Shallow Concentrated Flow

Shallow concentrated flow occurs after sheet flow but before open channel flow as sheet flow begins to concentrate along preferential flow paths in natural swales, small rills and gullies. Shallow concentrated flow is assumed not to have a well-defined channel and has flow depths less than 6 inches. Refer to Figure 4-4 for velocity versus slope for shallow concentrated flow. Figure 4-4 is based on the following equation and coefficients in Table 4-8.

$$Velocity = Velocity\ Coefficient\sqrt{S} \tag{4.4}$$

where:

Velocity = shallow concentrated flow velocity (ft./s)
Velocity Coefficient = based on Table 4-8 values
S = slope of land surface (ft./ft.)

Figure 4-4 Shallow Concentrated Flow Velocity

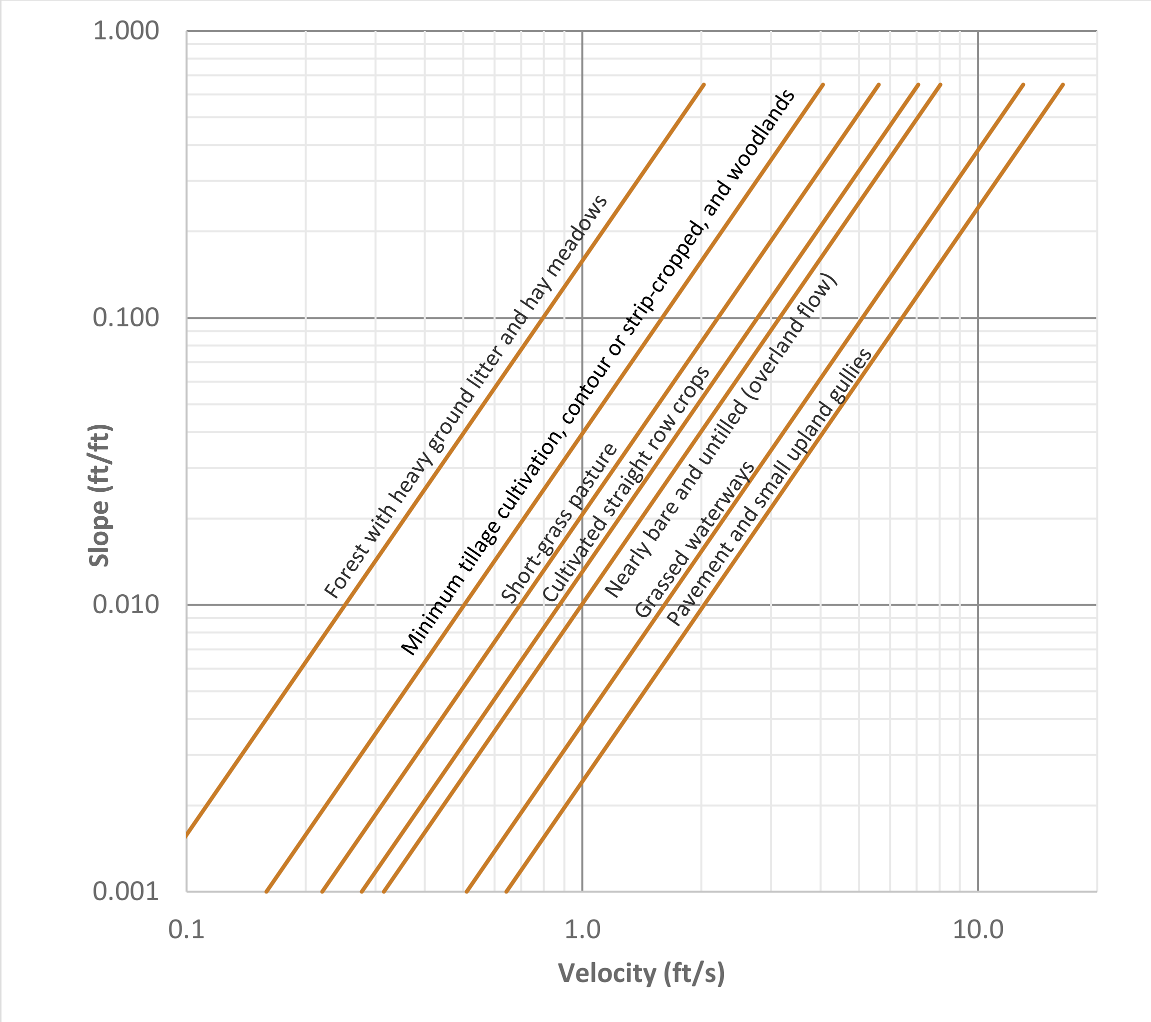


Table 4-8 Shallow Concentrated Flow Velocity Equation Coefficients

Flow Type	Assumed Depth for Shallow Flow (ft.)	Assumed Manning's n for Shallow Flow Condition	Velocity Coefficient for Equation (4.4)
Pavement and small upland gullies	0.2	0.025	20.328
Grassed waterways	0.4	0.05	16.135
Nearly bare and untilled (overland flow)	0.2	0.051	9.965
Cultivated straight row crops	0.2	0.058	8.762
Short-grass pasture	0.2	0.073	6.962
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	5.032
Forest with heavy ground litter and hay meadows	0.2	0.202	2.516

Open Channel Flow

Open channel flow is assumed to begin when shallow concentrated flow ends. Manning's equation should be used to estimate average flow velocity for the travel time. Open channel flow should be used for all flow in gutters, pipes, and manmade ditches and swales. Manning's equation is:

$$V = \frac{1.486R^{2/3}S^{1/2}}{n} \tag{4.5}$$

where V = velocity (ft./s)
R = hydraulic radius (ft.)
S = slope of the hydraulic grade line (ft./ft.)
n = Manning's roughness coefficient (refer to Chapter 5)

Storage Systems

The travel time through a temporary storage system is the time required to fill the SCM. An example of a temporary storage system is a small area of porous pavement surrounding a catch basin. The peak flow rate entering the storage system is first calculated. The time to fill the storage system is then calculated as the storage volume divided by the inflow rate.

In most cases the travel time through a body of water is assumed to be negligible and is typically ignored. This is because as water enters one end flow is assumed to exit at the same rate.

Minimum Time of Concentration

A minimum time of concentration of 5 minutes (or 0.1 hour) should be used based on the available rainfall data.

4.2.2 Rational Method

The Rational Method dates to 1889 and was developed to estimate the peak discharge from a storm event. The general form of the Rational Method equation is:

$$Q_p = CiA \tag{4.6}$$

where Q_p = Peak Flow, ft^3/s
C = Dimensionless runoff coefficient (Table 4-9)
i = Rainfall intensity, in/hr.
A = Drainage area, acre

A fundamental assumption inherent to the Rational formula is that rainfall intensity remains constant across the drainage area and over the duration required to drain the area (equal to the time of concentration). The time of concentration (t_c) is the time required for water to travel from the hydraulically most remote point of the basin to the point of interest (see previous section for method to determine t_c).

For watersheds that have long travel times, it is almost impossible to have a constant intensity over that time duration. Hence the Rational formula is only applicable to watersheds that have a relatively short time of concentration such as paved areas with curb, gutters and sewers. As a general rule of thumb, the Rational formula should only be applied to drainage areas smaller than 200 acres.

Runoff Coefficients

Runoff coefficients for recurrence interval discrete design storms up to and including a 10-year event are provided in Table 4-9. For peak flow calculations for larger storm events, the respective multiplier from Table 4-10 should be used with the runoff coefficient. When applying the multipliers (Table 4-10) for large storm events the adjusted runoff coefficient cannot exceed 1.0. If the drainage area contains varying amounts of different land cover or other abstractions, an area weighted average composite coefficient should be used.

Table 4-9 Runoff Coefficients (C) for Recurrence Intervals up to and including 10 years

Land Cover	Lower Limit	Upper Limit
Business		
Downtown	0.70	0.95
Neighborhood	0.50	0.70
Residential		
Light, 1 to 3 units per acre	0.35	0.45
Medium, 3 to 6 units per acre	0.50	0.60
Dense, 6 to 15 units per acre	0.70	0.80
Apartments	0.50	0.70
Industrial		
Light	0.50	0.80
Heavy	0.60	0.90
Pavement		
Asphalt	0.85	0.95
Brick	0.70	0.85
Concrete	0.90	0.95
Drives and Walks	0.75	0.95
Gravel	0.85	0.85
Earth Shoulders	0.50	0.50
Grass Shoulders	0.25	0.25
Median Area, turf	0.25	0.30
Railroad yard	0.30	0.40
Roofs	0.75	0.95
Parks		
Cemeteries	0.10	0.25
Playgrounds	0.20	0.40
Lawns, sandy soil		
Flat, less than 2%	0.05	0.10
Average, 2 – 7%	0.10	0.15
Steep, 7% or more	0.15	0.20
Lawns, heavy soil		
Flat, less than 2%	0.13	0.17
Average, 2-7%	0.18	0.22
Steep, 7% or more	0.25	0.35
Cultivated Land		
Sandy soil	0.25	0.35
Heavy soil	0.50	0.60
Natural		
Meadows & Pasture Land	0.25	0.35
Woodland & Forest	0.10	0.20

Table 4-10 Runoff Coefficient Recurrence Interval Multiplier

Recurrence Intervals	Runoff Coefficient (C) Multiplier
≤ 10 years	1.00
25 year	1.10
50 year	1.20
100 year	1.25

Rainfall Intensity

Precipitation frequency data is necessary to use the Rational method. Refer to Section 4.1.1 for information. The duration used to determine the rainfall intensity should equal the time of concentration for the drainage area.

4.2.3 Modified Rational Method

The Modified Rational Method has historically been used to size detention basins. The method employs a series of trapezoidal shaped hydrographs created from different storm durations. The allowable discharge rate is subtracted from each of the runoff volumes. The critical storm duration is the one which yields the greatest difference in volume between the post development hydrograph and the allowable discharge rate. The required storage volume is calculated based on the critical storm duration.

To avoid the iterative approach commonly used, regression equations describing the critical storm duration as a function of the allowable release rate and Rational coefficient (describing the land cover) were developed, refer to Figure 4-5. Defining the critical storm duration eliminates the iterative solution.

The critical storm duration is calculated using the applicable equation below. The selection of the 10- or 100-year recurrence interval is a function of the size of the drainage area as discussed in Chapter 2.

$$D_{10} = 30.9\left(\frac{Q_R}{C}\right)^{-0.979} \tag{4.7}$$

$$D_{100} = 49.988\left(\frac{Q_R}{C}\right)^{-0.984} \tag{4.8}$$

where D_{10} = critical storm duration for the 10-year event, min
D_{100} = critical storm duration for the 100-year event, min
C = Rational Coefficient, dimensionless, refer to Table 4-9
Q_R = peak allowable discharge rate, cfs/acre

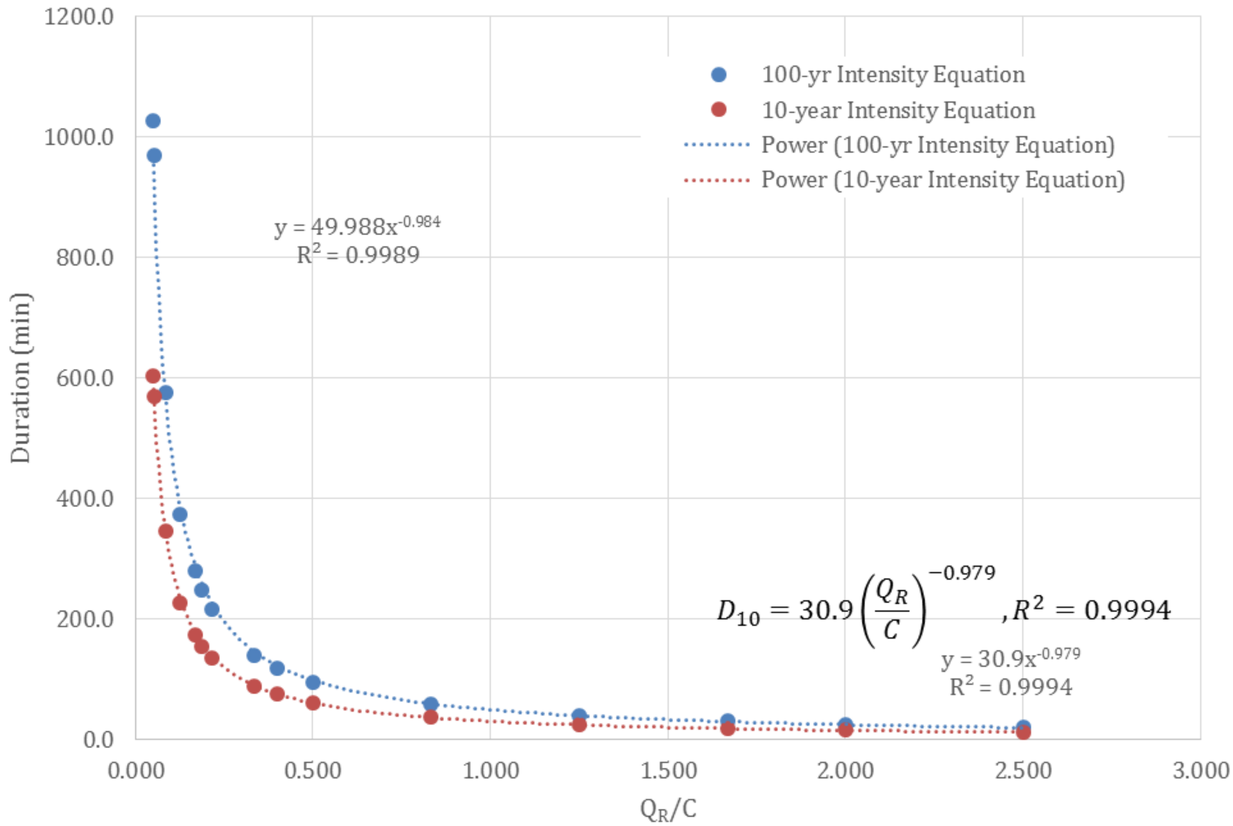


Figure 4-5 Critical Storm Duration Regression

The required storage volume is then calculated using the following equation:

$$V_n = 60.5 * D_n * C * A * I - 60 * D_n * Q_R * A \tag{4.9}$$

where A = tributary drainage area, acre
C = Rational Coefficient (dimensionless), refer to Table 4-9
D_n storm duration for the n-year event, min (critical storm duration)
I rainfall intensity (inches per hour) based on time of concentration
Q_R = peak allowable discharge rate, cfs/acre
V_n = required detention volume for the n-year event, ft^3

4.2.4 Small Storm Hydrology

Trends in environmental hydrology have highlighted the importance of managing small, frequent storms - the approximately 99% of events less than a 2-inch rainfall depth - for both water quality and receiving channel stability (WEF and ASCE, 1998; Pitt R. , 1999). Small Storm Hydrology is a widely used method for the calculation of stormwater runoff volume for rainfall depths typical of retention (i.e., groundwater recharge or volume reduction) or water quality treatment criteria (i.e., 0.5 – 1.5 in) (Claytor & Schueler, 1996; SEMCOG, 2008; CALTRANS, 2015) and has been suggested by MDEQ as a method to determine the runoff volume that must be managed to meet the City's water quality treatment performance standards (MDEQ, 2014). A primary reason for the use of small storm hydrology is that the curve number method is not intended to be used for small rainfall events (Hawkins, Ward, Woodward, & Mullem, 2009; Claytor & Schueler, 1996). Small storm hydrology can be expressed as:

$$V_{runoff} = Rv * P * A \qquad (4.10)$$

where V_{runoff} = volume of runoff
Rv = volumetric runoff coefficient
P = rainfall depth
A = area

See Chapter 2 for a discussion of the City's water quality performance standards. The area value is equal to a project's regulated area as discussed in Chapter 2.

The volumetric runoff coefficient (Rv) for a land use is an empirically derived value that indicates the fraction of rainfall that is converted into runoff for that land use (Pitt R. E., 1987; Schueler, 1987). The most common ways Rv is determined include: (1) from a linear regression equation in which Rv is a function of the impervious area within the contributing drainage area; (2) a polynomial equation as a function of impervious area, (3) from look-up tables in which Rv is a function of both land use/cover and rainfall depth, and (4) a hybrid approach from the first 3 methods. The regression equation methods (Methods 1 and 2) provide a quick and easy approach appropriate for conceptual design applications. Final design applications should utilize the tabular method (Method 3).

Method 1: Linear Regression Equation for Rv

A common small storm hydrology method for predicting runoff volume establishes Rv as a linear relationship with impervious area (Schueler, 1987). This is sometimes referred to as the *Simple Method* or *Shortcut Method*.

$$Rv = 0.05 + 0.009 * I \qquad (4.11)$$

where Rv = volumetric runoff coefficient
I = percent impervious (0-100)

The equation is based on data from the Nationwide Urban Runoff Program (NURP) (U.S. Environmental Protection Agency, 1983). This relationship is used by several states or municipalities to estimate runoff volume for smaller storms (CALTRANS, 2015), and has been used in the Simple Method water quality model for estimating pollutant loads since its introduction by Schueler (Schueler, 1987). Other relationships between Rv and impervious area have been proposed but are used less widely (CALTRANS, 2015).

Method 2: Polynomial Regression Equation for Rv

A third order polynomial equation was also developed from the NURP data (WEF and ASCE, 1998). A key point illustrated by the third order polynomial equation is the degree to which Rv may not be a linear function.

$$Rv = 0.858\left(\frac{I}{100}\right)^3 - 0.78\left(\frac{I}{100}\right)^2 + 0.774\left(\frac{I}{100}\right) + 0.04 \qquad (4.12)$$

where Rv = volumetric runoff coefficient
I = percent impervious (0-100)

Method 3: Tabular Method for Rv

A more refined version of small storm hydrology utilizes Rv's that both: (1) are specific to the individual land uses or land covers present in the tributary drainage area; and (2) change with the depth of the rainfall event. Refer to Table 4-11 for a look-up table that lists the Rv's separately based on land cover and rainfall depth. This approach allows for subtle or not-so-subtle differences in runoff generation among impervious land surfaces (e.g., pitched roofs vs flat roofs vs smooth pavement vs rough pavement) or among pervious land surfaces (e.g., sandy vs clayey soils). The runoff volume from the impervious area is calculated from an area-weighted Rv:

$$Rv = \frac{Rv_1 * A_1 + Rv_2 * A_2 + Rv_3 * A_3 + \ldots + Rv_n * A_n}{A_{total}} \qquad (4.13)$$

where Rv_n = volumetric runoff coefficient for a land cover
A_n = tributary drainage area for a land cover

For rainfall depths that fall between values in the table, the Rv is determined by interpolation.

Method 4: Hybrid Approach for Rv

Several states and municipalities have taken a hybrid approach between the previously discussed methods to simplify implementation but still provide some level of specificity or flexibility where it supports program goals. In such cases, all impervious areas are typically assigned a single Rv value (e.g., Rv = 0.9 or Rv = 0.95) for the target rainfall event depth, whereas pervious areas may have varied Rv values to reflect soil type.

Table 4-11 Volumetric Runoff Coefficients

Rainfall (in)	Volumetric Runoff Coefficients, Rv						
	Impervious Areas				Pervious Areas		
	Connected Flat Roofs/ Large Unpaved Parking Areas	Connected Pitched Roofs	Directly Connected Large Imperv. Areas	Small Imperv. Areas and Uncurbed Roads	Sandy Soils (HSG A)	Silty Soils (HSG B)	Clayey Soils (HSG C&D)
0.5	0.75	0.94	0.97	0.62	0.02	0.09	0.17
1.0	0.84	0.97	0.97	0.67	0.02	0.12	0.21
1.5	0.88	0.99	0.99	0.77	0.05	0.15	0.24

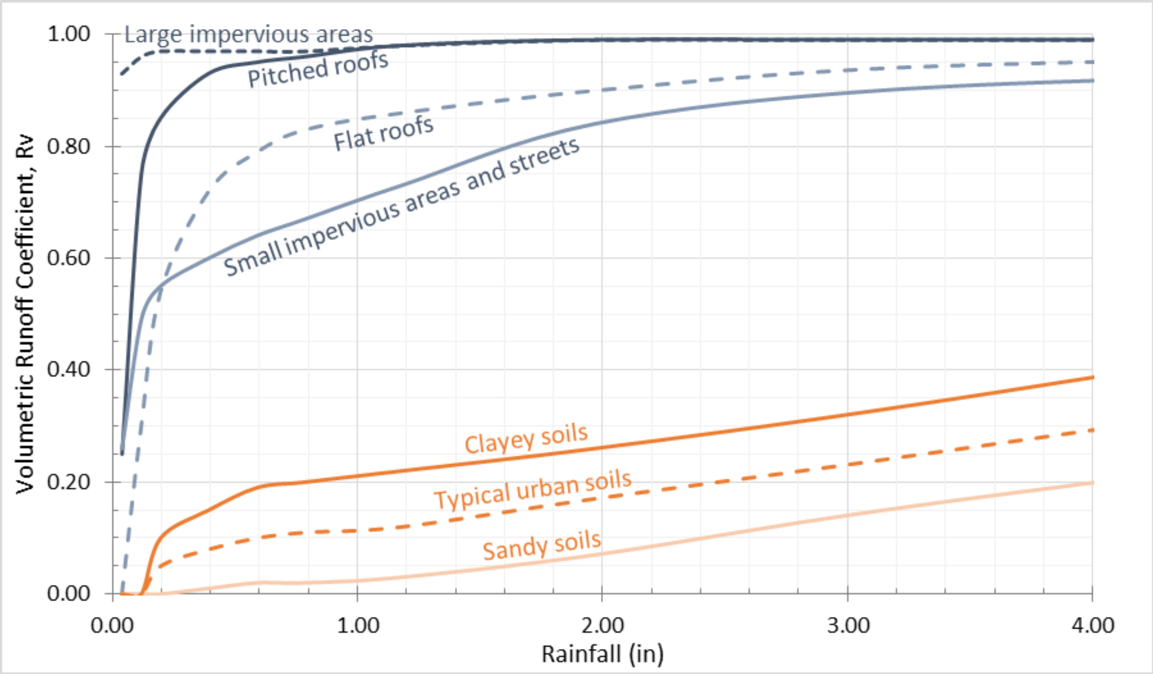


Figure 4-6 Volumetric Runoff Coefficients (Pitt 1987)

4.2.5 Curve Number Method

The Natural Resources Conservation Service (NRCS) curve number (CN) method may be used to estimate the direct runoff volume from a storm event. When coupled with a unit hydrograph approach, the curve number method may be used to estimate a complete runoff hydrograph including runoff rate and volume over the duration of the storm event. The fundamental curve number runoff equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \; for \; P > I_a \qquad (4.14)$$

$$Q = 0 \; for \; P \leq I_a \qquad (4.15)$$

where Q = depth of runoff, in
P = depth of rainfall, in
I_a = initial abstraction, in
S = maximum potential retention, in

The maximum potential retention (S) is related to the curve numbers (CN) by:

$$CN = \frac{1000}{(10 + S)} \qquad (4.16)$$

where CN = Curve Number, dimensionless (Table 4-12)
S = maximum potential retention, in

Initial Abstraction

An empirical relationship between initial abstraction (I_a) and potential maximum retention (S) is often assumed as:

$$I_a = \lambda S \qquad (4.17)$$

Where λ (lambda) is historically assumed to be a constant and has been customarily taken as λ=0.20.

After substituting the assumed initial abstraction coefficient (λ=0.20) the general rainfall-runoff relationship is:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \; for \; P > I_a \qquad (4.18)$$

Researchers have reevaluated the relationship and suggest a value of λ of 0.05 provides a more appropriate fit for general applications (Hawkins & Khojeini, 2000; Jiang, 2001). A value of λ = 0.05 is allowed, however the CN coefficients must be adjusted from the standard CN tables which assume λ=0.20. Converting standard CN's to an equivalent CN based on λ=0.05 may be done with the following equation (Hawkins, Ward, Woodward, & Mullem, 2009):

$$CN_{0.05} = \frac{100}{\left(1.879\left(\frac{100}{CN_{0.20}} - 1\right)^{1.15} + 1\right)} \qquad (4.19)$$

where $CN_{0.05}$ = CN based on λ=0.05

$CN_{0.20}$ = CN based on λ=0.20 (Table 4-12)

Hydrologic Soil-Cover Complexes

A combination of a hydrologic soil group and a land use and treatment class is a hydrologic soil-cover complex. Refer to Table 4-12 for acceptable curve number coefficients for each land cover description. The hydrologic soil group should be determined by the soil data from the NRCS's Web Soil Survey. Chapter 6, Soil, Aggregate and Water, contains additional information on hydrologic soil groups.

Table 4-12 Curve Number Coefficients

Land Cover Description [(1)]	Hydrologic Soil Group			
	A	B	C	D
Commercial and Business (est. 85% impervious)	89	92	94	95
Industrial (est. 72% impervious)	81	88	91	93
Residential				
1/8-acre lot (town houses) (est. 65% impervious)	77	85	90	92
1/4-acre lot (est. 38% impervious)	61	75	83	87
1/3acre lot (est. 30% impervious)	57	72	81	86
1/2-acre lot (est. 25% impervious)	54	70	80	85
1-acre lot (est. 20% impervious)	51	68	79	84
2-acre lot (est. 12% impervious)	46	65	77	82
Developing; Newly Graded (pervious only, no vegetation)	77	86	91	94
Urban Open Space (lawns, parks, golf, cemeteries)				
Poor (grass cover <50%)	68	79	86	89
Fair (grass cover 50% to 75%)	49	69	79	84
Good (grass cover >75%)	39	61	74	80
Urban; Paved Parking, Roofs, Driveways (excl. ROW); 100% impervious	98	98	98	98
Streets and Roads				
Paved; curbs and storm sewers (excl. ROW)	98	98	98	98
Paved; open ditches (incl. ROW)	83	89	92	93
Gravel (incl. ROW)	76	85	89	91
Dirt (incl. ROW)	72	82	87	89
Rural, farmstead-building, lanes, driveways, and surrounding lots	59	74	82	86
Natural				
Brush, Forb, Grass Mix, Poor vegetation cover	48	67	77	83
Brush, Forb, Grass Mix, Fair vegetation cover	35	56	70	77
Brush, Forb, Grass Mix, Good vegetation cover [(2)]	30	48	65	73
Woods, Poor vegetation cover, Destroyed by Grazing or Burning	45	66	77	83
Woods, Fair vegetation cover, Grazed but not Burned, some forest liter	36	60	73	79
Woods, Good vegetation cover, Protected from Grazing, litter/brush cover	30	55	70	77
Mix; Woods-Grass Combination, Orchard, Tree Farm; Poor	57	73	82	86
Mix; Woods-Grass Combination, Orchard, Tree Farm; Fair	43	65	76	82
Mix; Woods-Grass Combination, Orchard, Tree Farm; Good	32	58	72	79
Water	100	100	100	100

(1) All curve numbers assume λ=0.20.

(2) Assumed predevelopment condition.

Subwatershed Delineation

The time of concentration for pervious areas is typically much larger than disconnected impervious. Similarly, the time of concentration for disconnected impervious is much larger than connected impervious areas. Since the times of concentration are so disparate combining them into the same watershed greatly underestimates the combined discharge peak because the discharge characteristics of the impervious area(s) are sharply muted. Therefore, the pervious, directly connected impervious and disconnected impervious areas should all be considered as separate subwatersheds.

Weighted Runoff (Q) verse Weighted CN

Calculating the runoff from subwatersheds having more than one hydrologic soil-cover complex may be done one of two ways. A common approach is to calculate a weighted average CN based on drainage area first and then use the weighted average CN to compute runoff. This method requires less work than a weighted runoff approach, however, where differences in CN for the various subwatersheds are large, this method leads to inaccurate runoff volumes. *A weighted CN approach is not allowed.*

The preferred approach is to compute the runoff independently from each subwatershed using the unique hydrologic soil-cover complex. A weighted runoff volume is then computed from each subwatershed.

Small Storms

Curve numbers were originally developed based on annual flood flows and the most common application was to determine a design discharge for 25-year events to probable maximum floods. The method generally does not work well for when the runoff is a small fraction of the rainfall, (i.e., CN's are low or rainfall values are small). As a general guideline, runoff from rainfall events smaller than 1 to 1.5 inches should be computed with a method other than the CN method.

Unit Hydrograph

The Curve Number method may be coupled with a unit hydrograph to estimate the resulting runoff hydrograph from an event. Refer to the *National Engineering Handbook Part 630 Hydrology*, Chapter 16 for a complete discussion of unit hydrograph application (Natural Resources Conservation Service, 2007). Attention should be paid to the assumed peak rate factor. The default value of 484 may not be applicable and some computer programs do allow altering the value.

4.2.6 EPA SWMM

USEPA's Storm Water Management Model (SWMM) is an acceptable model for computing and routing runoff from a site. SWMM is a public domain software. It may be used for single event or continuous simulations. SWMM includes a specialized "LID Controls" editor to model seven different types of green stormwater infrastructure practices: permeable pavement, rain gardens (bioretention practices), green roofs, street planters, rain barrels, infiltration trenches, and vegetative swales. Note because the LID Controls are simplified representations of the green stormwater infrastructure practices, they may not be able to accurately model every design scenario.

The EPA SWMM engine is incorporated into several third-party computer modeling software packages. When using SWMM: (1) the most recent version of the EPA software shall be used; and (2) any models submitted shall be the EPA version of SWMM, not a version exclusive to a third-party software package.

Additional information and resources are available on the USEPA's SWMM website.

4.3 References

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5. Drainage Conveyance

The purpose of this chapter is to provide standards and criteria to ensure the safe and effective conveyance of stormwater through a storm drainage system in a manner consistent with the protection of public health, safety and welfare; the protection and function of infrastructure and other improvements; and maintenance or improvement of water and environmental quality in the City of Detroit and its surface waters.

This chapter provides guidance on the conveyance of stormwater drainage including open channels, culverts, and storm sewers. Aspects of stormwater drainage design such as pavement drainage, gutter flow calculations, inlet sizing, pipe and channel sizing, and hydraulic grade line calculations are included.

Contents

5.1 Stormwater Conveyance Systems

Stormwater conveyance systems may consist of natural streams, channels, vegetated swales, open ditches, closed conduits or a combination of methods to convey stormwater. The applicant must construct drainage facilities in accordance with the City's minimum specifications presented in this manual. Other standards may apply depending on location of the outlet.

5.2 Hydraulic Calculations

5.2.1 Introduction

Hydraulic calculations are used to size conduits or open channels to handle the design flows calculated from hydrologic calculations. The hydraulic capacity of a storm sewer conduit or culvert can be calculated for the two types of conditions typically referred to as gravity and pressure flow. Open channel facilities are evaluated considering only gravity flow.

When the discharge point is not submerged, a flow depth should be calculated at a known control section to establish a starting elevation. The hydraulic grade line is then projected from the starting elevation to the upstream manhole. Computations continue from downstream to upstream for typical subcritical flow conditions. In cases of supercritical flow conditions, calculations start upstream and go in the downstream direction. Pressure flow calculations may be used at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines is not entirely correct, since backwater and drawdown conditions can exist, but is generally reasonable. It is also usually appropriate to assume the hydraulic grade calculations begin at the crown of the outlet pipe for simple, non-submerged systems. If additional accuracy is needed, as with very large conduits or where the result can greatly affect design, backwater and drawdown curves should be developed.

5.2.2 Tailwater

For most design applications where the flow is subcritical, the tailwater will either be above the crown of the outlet or can be between the crown and critical depth. To determine the EGL, begin with either the tailwater elevation or $(d_c + D)/2$, whichever is higher; add the velocity head for full flow; and proceed upstream, adding appropriate losses (e.g., exit, friction, junction, bend, entrance).

An exception to the above procedure is an outfall with low tailwater. In this case, a water surface profile calculation would be appropriate to determine the location where the water surface will either intersect the top or end 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 tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, consider the coincidental probability of two events occurring at the same time. A short duration storm which causes peak discharges on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. Refer to Table 5-1 for tailwater recurrence interval selection (MDOT, 2006).

Table 5-1 Frequency of Coincidental Occurrences

Area Ratio (Receiving System to Contributing System)	10 Percent (10-Year) Chance Storm in Contributing System Receiving System Recurrence Interval	2 Percent (50-Year) Chance Storm in Contributing System Receiving System Recurrence Interval
10,000 to 1	100% (1-yr)	50% (2-yr)
1,000 to 1	50% (2-yr)	20% (5-yr)
100 to 1	20% (5-yr)	10% (10-yr)
10 to 1	10% (10-yr)	4% (25-yr)
1 to 1	10% (10-yr)	2% (50-yr)

5.3 Open Channels

Open channels are surface drainage features designed, constructed and maintained to convey stormwater runoff without allowing channel erosion. Important design parameters for open channels include return period of the design event, channel slope, channel geometry, vegetation type and freeboard. Roadside ditches are considered open channels.

5.3.1 Design Storm

Open channels, including roadside ditches, shall be designed for the 10 percent chance (10-year) storm. In special cases the channel shall be designed to carry the 2 percent chance (50-year) frequency event. Design conditions should not cause erosion, sedimentation or overbank flooding.

Other considerations include inconvenience, hazards, and nuisances to pedestrian traffic and building which are located within the splash zone. These considerations should not be underestimated and, in some locations (such as commercial areas), may assume major importance.

5.3.2 Slope and Hydraulic Gradient

The hydraulic grade line must be a minimum of 1.0 ft. below the edge of the road shoulder or gutter grade.

The velocity shall be between 2 and 8 feet per second based on the design storm flow condition.

The minimum allowable grade is 0.1 percent. A minimum grade of 0.3 percent is recommended. Channel slopes shall be stabilized against erosion by either adequate vegetation or protective armoring.

5.3.3 Channel Shape

Trapezoidal or parabolic cross sections shall be used, with a minimum 2-foot bottom width for trapezoidal channels is recommended. Triangular or ‘V’ shaped channels may be used.

Channel side slopes shall be stable throughout the length of the channel. A maximum side slope 1V:3H shall be used to ensure slope stability. Side slopes of 1V:4H or flatter are recommended to facilitate maintenance.

Two-stage channel design should be considered in the design of channels with large cross sections to help with channel stability. As a rule-of-thumb, channels with design flows greater than 100 ft3/s may be considered to have large cross sections. Channels with base flow conditions or other natural features should also be designed as two-stage channels. Refer to the NRCS National Engineering Handbook Part 654 for information on two-stage channel design (NRCS, 2007).

Figure 5-1 Two-stage channel

5.3.4 Stable Channel Design

Channels shall be designed to be stable; meaning that the channel lining effectively resists the erosive forces of the flow for design conditions. Attention should be paid to flow around a bend in a channel, at culvert entrances and exits, and where concentrated flow enters the channel (e.g. a pipe discharging into the channel).

Lining materials may be classified as flexible or rigid. Flexible lining materials, such as grass and riprap, are limited in the erosive forces they can sustain without damaging the channel. A rigid lining, such as concrete, can typically provide greater erosion resistance. A temporary or biodegradable (net or mat type) channel liner may be necessary to prevent erosion until vegetation seeding becomes established and the channel lining is stabilized. Table 5-2 provides guidelines on channel stabilization treatments.

Table 5-2 Channel Stabilization Guidelines

Lining	Grade
Seed and Mulch	≤ 0.5%
Standard Mulch Blanket	0.5% to 1.5%
High Velocity Mulch Blanket or Sod	1.5% to 3.0%
Turf Reinforcement Mat or Cobble	3.0% to 6.0%
Specific Design Required	> 6%

(MDOT, 2006)

Additional recommended information on stable channel design includes:

- Michigan Nonpoint Source Best Management Practices Manual by Michigan DEQ (MDEQ, 2017)
- Design of Roadside Channels with Flexible Linings by FHWA (FHWA, 2005)

5.4 Culverts

Culverts are contained within open channels and are used to safely convey water from one side of a roadway, driveway or embankment to the other.

Culverts shall be sized using the nomographs presented in the MDOT *Drainage Manual* or the FHWA report *Hydraulic Design of Highway Culverts,* or using the following approved computer software programs:

a) HY8 (FHWA Culvert Analysis Software)

b) HEC-RAS (Hydraulic Engineering Center – Riverine Analysis Systems)

As presented in the FHWA report, the two basic types of culvert control sections are inlet and outlet control. The control section for inlet control is just inside the entrance, and critical depth occurs at or near this location. The control section for outlet control is located at the barrel exit or downstream from the culvert. Either partially full subcritical flow or full pipe pressure flow conditions can occur.

If inlet control exists, the culvert barrel could possibly carry more flow than the inlet will accept, and if this is the case, a tapered inlet could be used to increase capacity up to the outlet capacity. If outlet control exists, the culvert barrel would have to be increased to add capacity. Once a culvert size has been determined from the nomographs, reductions may be warranted if storage occurs at the culvert embankment.

5.4.1 Application Categories

For consistency, culvert applications are divided into two major categories, cross drains and side drains:

Cross Drain

A cross drain is a culvert placed transversely under roadway sections, with end walls or some other end treatment. Because cross-drain installations are normally under pavement, they shall have at least premium joint-RCP to prevent soil migration. Leaking joints can cause uneven and differential settling of road surfaces or adjacent buildings.

Side Drain

This culvert is generally a pipe used longitudinally in roadway ditches under driveways or graded connections.

5.4.2 Design Storm

Culverts shall be designed, constructed and maintained to convey the same design storm as identified for the open channel.

5.4.3 Allowable Headwater

The allowable headwater elevation can be established from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. In general, the constraint that gives the lowest allowable headwater elevation should establish the basis for hydraulic calculations. The following criteria should be considered:

Backwater Impacts and Flood Elevations

Non-damaging or permissible upstream flooding elevations (e.g., existing buildings or flood insurance rate map elevations) should be identified and headwater for the design discharge kept a minimum of 1 ft. below. Level pool backwater conditions should be evaluated upstream from the culvert to ensure that building flooding does not occur for the 100-year, 24-hour design storm.

Maximum Allowable Headwater

Headwater depth for cross-drain design discharge shall not exceed a height greater than 1 ft. below the edge of the shoulder of a road. Headwater depth for side-drain discharge shall not exceed the height of the near edge of pavement for driveway culverts or bicycle pathways.

5.4.4 Design Tailwater

The hydraulic conditions downstream of the culvert site should be evaluated to determine a tailwater depth for the design discharge. This is a crucial factor in determining culvert capacity under outlet control conditions. The determination shall consider downstream constraints, obstructions, or other hydraulic features that may create backwater at the culvert outlet. For culverts that discharge to an open channel, the normal depth of flow in the channel must be determined. If the culvert outlet is operating in a free-fall condition (e.g., a cantilever pipe), the critical depth and equivalent hydraulic grade line shall be determined. Guidance for performing these evaluations is available in the *Hydraulic Design of Highway Culverts* (FHWA, 2012) report.

5.4.5 End Treatments

End treatment facilities shall be consistent with hydraulic requirements, and consider bank stability, safety, and costs. Entrance loss coefficients (k_e) summarized in Table 5-3 shall be used in design.

5.4.6 Velocity Limitations

Both minimum and maximum velocities shall be considered when designing a culvert. A minimum velocity of 2.5 feet per second for full culvert flow shall be used to ensure a self-cleaning condition during partial depth flow.

Culvert outlet velocities shall be calculated to determine the need for erosion protection at the culvert exit. Culverts usually result in outlet velocities which are higher than the open channel velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. Outlet velocities less than 6.0 ft/s will generally not require special treatment if a headwall or end section is used.

5.4.7 Length, Slope, and Size

The length and slope of a culvert shall be based on the bottom of the stream or channel being conveyed, the geometry of the roadway embankment, and the skew angle of the culvert. A culvert slope near the existing topography should be chosen. The minimum culvert size is 12 inches.

Table 5-3 Culvert Entrance Loss Coefficients

Type of Structure and Design of Entrance	Entrance Coefficient, k_e
Pipe, concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square-cut end	0.5
Headwall or headway 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 [a]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or sloped-tapered inlet	0.2
Pipe or Pipe Arch, Corrugated Metal	
Projecting from fill (no headway)	0.9
Headway or headway and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End section conforming to fill slope [a]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headway parallel to embankment (no wingwalls)	
Square-edged on three edges	0.5
Rounded on 3 edges to radius of 1/12-barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° or 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12-barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

(a) End section conforming to fill slope, made of either metal or concrete, is the section commonly available from manufacturers. From limited hydraulic tests, the sections are equivalent in operation to a headway in both inlet and outlet control. End sections that incorporate a closed taper in their design have a superior hydraulic performance.

5.5 Gutter Flow

5.5.1 Water Spread

The top width of the open channel flow in the gutter is considered the spread. In general, the water spread should be limited to a specified width for the selected design frequency. For storms of greater magnitude, the spread can be allowed to use most of the pavement as an open channel.

Table 5-4 Design Frequency and Spread for Roadways

Roadway Classification		Design Frequency	Design Spread
High Volume	<45 mph	10-yr	Shoulder + 3 ft.
	>45 mph	10-yr	Shoulder
	Sag point	50-yr	Shoulder + 3 ft.
Collector	<45 mph	10-yr	½ driving lane
	>45 mph	10-yr	Shoulder
	Sag point	10-yr	½ driving lane
Local Streets	Low ADT	5-yr	½ driving lane
	High ADT	10-yr	½ driving lane
	Sag point	10-yr	½ driving lane

The spread width may be adjusted if warranted by an assessment of the costs vs. risks. If the above spread requirement results in very close inlet spacing (i.e., 100 ft. or less), then alternative drainage interceptors may be considered.

5.5.2 Gutter Flow Calculations

Introduction

Gutter flow calculations are necessary to relate the quantity of flow (*Q*) in the curbed channel to the spread of water on the shoulder, parking lane, or traveled way section. This section discusses uniform cross slope roadways and composite gutter sections. Composite gutter sections have a greater hydraulic capacity and are therefore preferred. Figure 5-2 presents schematics of typical gutter sections. If one of the swale sections illustrated in Figure 5-2 are proposed, see HEC-22 (FHWA, 2013) for procedures for calculating spread.

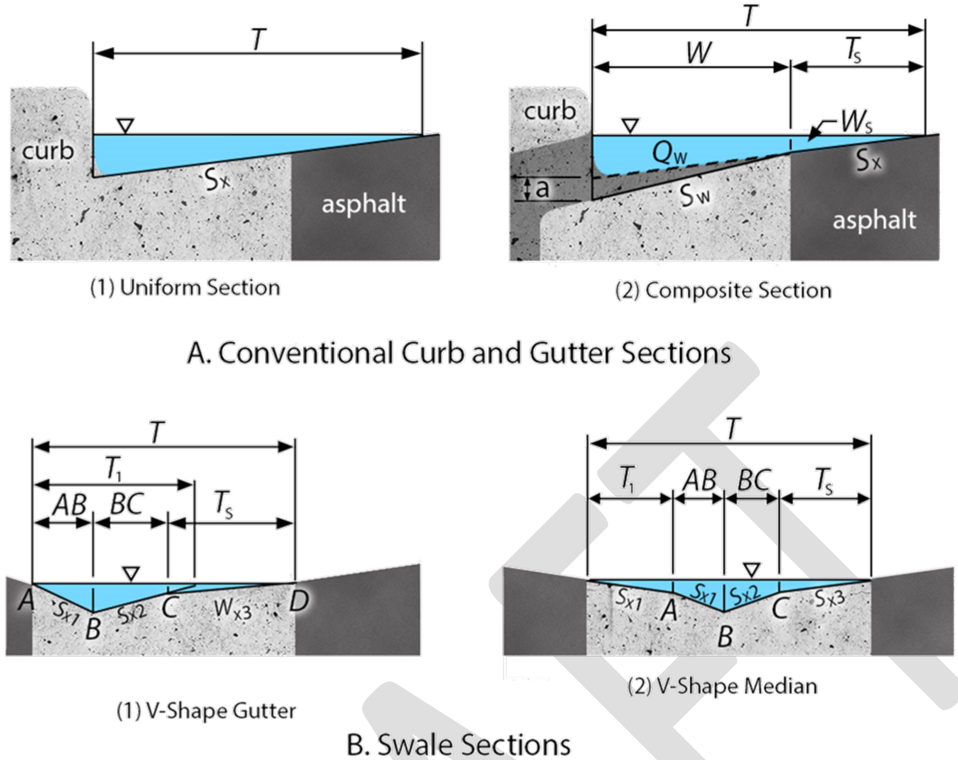


Figure 5-2 Typical Curb and Gutter Sections

Capacity Relationship

A modification of Manning's equation can be used for computing flow in triangular channels with Equation (5.1) or in terms of the flow width with Equation (5.26)

$$Q = (K_u/n)S_x{}^{1.67}S_L{}^{0.5}T^{2.67} \tag{5.1}$$

$$T = \left[(Qn)/\left(K_u S_x{}^{1.67}S_L{}^{0.5}\right)\right]^{0.375} \tag{5.2}$$

where Q = flow rate, ft^3/s
K_u = 0.56
n = Manning's coefficient, refer to Section 5.12
S_X = cross slope, ft./ft.
S_L = longitudinal slope, ft./ft.
T = width of flow (spread), ft.

Composite Gutter Section Procedure

The design of a composite gutter section requires the consideration of flow in the depressed segment of the gutter, Q_w. The equations provided below can be used to determine the flow in a width of gutter in a composite cross section, *W*, less than the total spread, *T*.

$$E_o = 1 \Big/ \left\{ 1 + \frac{S_W/S_X}{\left[1 + \frac{S_W/S_X}{(T/W) - 1}\right]^{2.67} - 1} \right\} \qquad (5.3)$$

$$Q_w = Q - Q_s \qquad (5.4)$$

$$Q = Q_s/(1 - E_o) \qquad (5.5)$$

where E_o = ratio of flow in the depressed section to the total flow,
S_X = cross slope of the gutter, ft./ft.
$S_W = S_X + a/W$, refer to Figure 5-6
T = top width of water surface, ft.
W = gutter depression width, ft.
Q_W = flow rate in the depressed section of the gutter, ft^3/s
Q = gutter flow rate, ft^3/s
Q_S = flow capacity of the gutter section above the depressed section, ft^3/s

5.6 Inlets

5.6.1 Inlet Types

Grate inlets and the depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets should be bicycle safe where used on roadways that allow bicycle travel.

Inlets used to collect surface water from pavement may be grouped into four major classes. These classes include: grate inlets, curb opening inlets, combination inlets, and slotted drain inlets. This section discusses the several types of inlets used and provides guidelines on the use of each type.

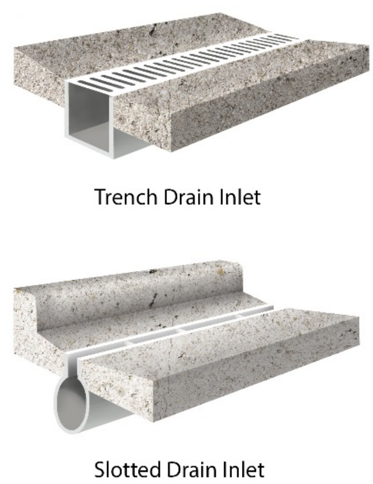
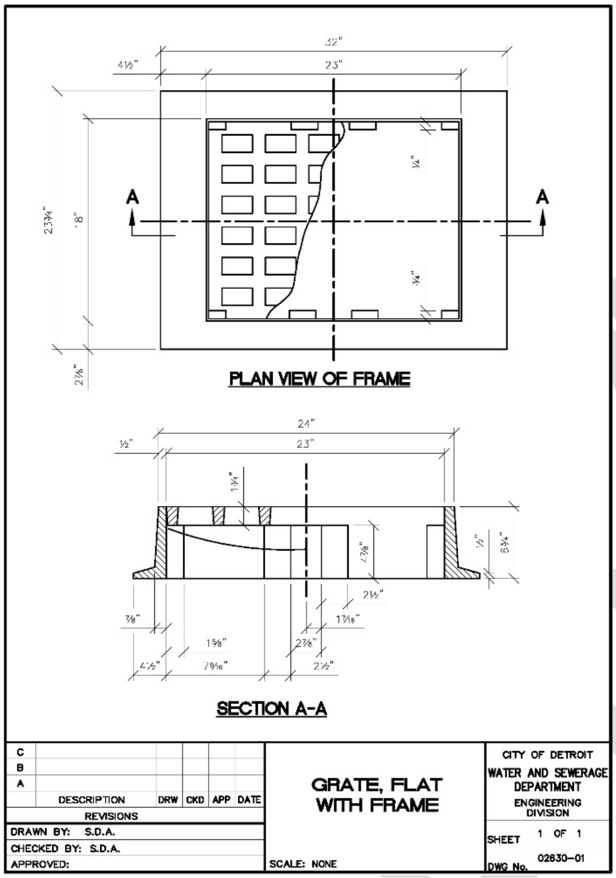


Figure 5-3 Inlet Types

Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Because they are susceptible to clogging with debris, the use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special-design (oversize) grate inlets can be used at major sag points if sufficient capacity is provided for clogging. Otherwise, flanking inlets are needed.

A flat grate inlet is the standard inlet type in Detroit. Three grate choices are available: a standard grate and two throttled catch basin covers.



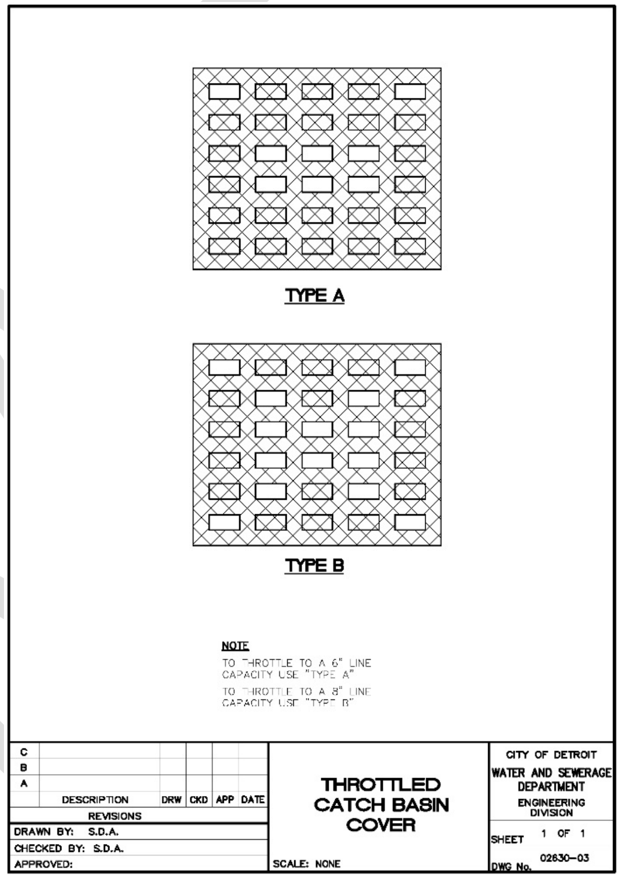


Figure 5-4 Standard Grates

Curb-Opening Inlets

These inlets provide openings in the curb covered by a top slab. Curb-opening inlets are preferred at sag points because they can convey significant quantities of water and debris. They may also be a viable alternative to grates in many locations where grates may be hazardous for pedestrians or bicyclists. They are generally not the first choice for use on continuous grades because of their poor hydraulic capacity.

Combination Inlets

Several types of combination inlets are in use. Curb-opening and grate combinations are common, some with the curb opening upstream of the grate and some with the curb opening adjacent to the grate. The gutter grade, cross slope, and proximity of the inlets to each other are significant factors when selecting this type of inlet. Combination inlets may be desirable in sags because they can provide additional capacity in the event of plugging.

Slotted Drain Inlets

These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs because the flow usually enters perpendicular to the slot. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets.

Slotted corrugated metal pipes may be used in median crossovers and, at times, in curb and gutter sections where large volumes of water need to be picked up. Slotted reinforced concrete pipe may be used as a median drain.

5.6.2 Inlet Location, Spacing and Capacity

Drainage inlets are sized and located to limit the spread of water on the roadway to allowable widths for the design storm as specified in Section 5.5.1. In addition, there are many locations where inlets may be necessary with little regard to contributing drainage area. Examples of such locations are all low points in the gutter grade or inlet spacing on continuous grades.

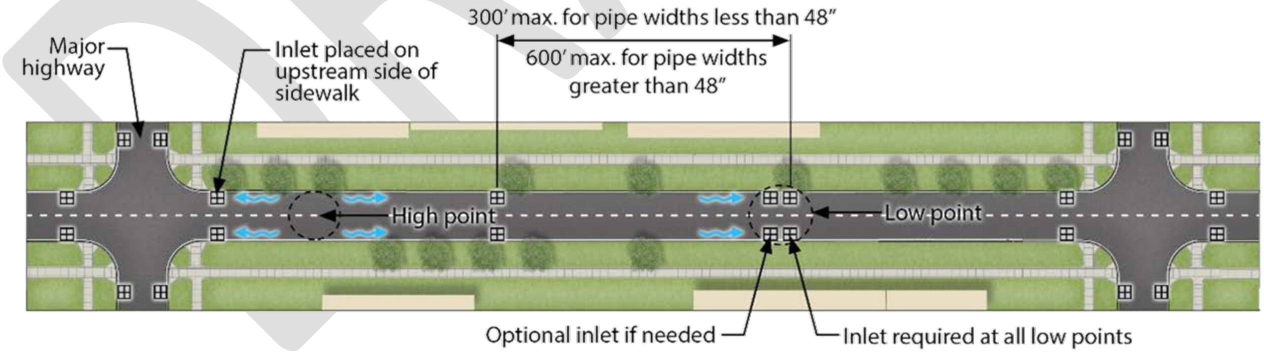


Figure 5-5 Example inlet locations

The following are guidelines for choosing inlet locations:

- Regardless of the results of the hydraulic analysis, inlets on grade should be spaced at a maximum of 300 ft. for 48 in. or smaller pipes.
- Inlets on grade should be spaced at a maximum of 600 ft. for pipes larger than 48 in.

- Inlets should be placed on the upstream side of bridge approaches.
- Inlets should be placed at all low points in the gutter grade.
- Inlets should be placed at points on either side of the low point that are 0.2 foot higher than the low point, or a maximum of 75 feet either side of the low point. Other alternatives, such as slotted drains, may be considered in lieu of additional catch basins. This applies in long sags.
- Inlets should be placed on both sides of cross streets that drain toward the roadway. Water should never be carried across intersections or crosswalks in valley gutters or troughs.
- Inlets should be placed on the upstream side of a driveway entrance, curb-cut ramp, or pedestrian crosswalk even if the hydraulic analysis places the inlet further downgrade or within the feature.
- Inlets should be placed upstream of median breaks.
- Inlets should be placed to capture flow from intersecting streets before it reaches the major highway.
- Flanking inlets in sag vertical curves are standard practice.
- Inlets should be placed to prevent water from sheeting across a roadway highway (i.e., place the inlet before a superelevation transition begins).
- Inlets should not be in the path where pedestrians walk.
- Inlets should be placed behind shoulders or back of sidewalks to drain low spots.
- For concrete pavements, catch basins should not be placed at spring points of street intersections as they interfere with the construction of the expansion joint at that location. They should be placed 10 feet either side of the spring line, or at the midpoint of the arc if detailed grades indicate that location as the low spot in the grade.
- The use of 24-inch-diameter catch basins should be limited to upstream ends of sewer runs where the run to the next drainage structure is 65 feet or less, and where the structure depth does not exceed 8 feet. Use 48-inch-diameter drainage structures for catch basins in all other locations.
- Do not locate drainage structures in line with a sidewalk ramp. Except where existing structures are being used, the location of the ramp takes precedence over the location of the drainage structures. Grades may need to be adjusted to accomplish this.

5.6.3 Spacing Process

Locate inlets from the crest and work downgrade to the sag points. The location of the first inlet from the crest can be found by determining the length of pavement and the area behind the curb sloping toward the roadway that will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel that will meet the design frequency and allowable water spread.

To space successive downgrade inlets, it is necessary to compute the amount of flow that will be intercepted by the inlet (Q_i) and subtract it from the total gutter flow to compute the bypass. The bypass from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the allowable water spread.

5.6.4 Curb-Opening Inlets

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. *Curb openings are also a very common approach to route drainage into a bioretention system and other GSI practices.*

Curb-Opening Inlets at Grade

The length of a curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}(S_L)^{0.3}\left[\frac{1}{nS_X}\right]^{0.6} \tag{5.6}$$

where L_T = curb-opening length required to intercept 100% of the gutter flow, ft.
K = 0.6
Q = peak flow, ft^3/s
n = Manning roughness coefficient, refer to Section 5.12
S_L = longitudinal slope of the gutter, ft./ ft.
S_X = cross slope of the gutter, ft./ft.

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections or for a continuously depressed gutter (composite gutter) can be found by substituting the equivalent cross slope (S_E) given by Equation (5.7) for the cross slope (S_X) in Equation (5.6). Refer also to Figure 5-6 for an illustration of a depressed curb opening cross section.

$$S_E = S_X + S'_W E_o \tag{5.7}$$

$$S'_W = \frac{a}{W} \tag{5.8}$$

where S_E = equivalent cross slope of a depressed curb opening, ft./ft.
S_X = cross slope of the gutter, ft./ft.
S'_W = cross slope of the gutter measured from the cross slope of the pavement ft./ft.
a = gutter depression depth, ft.
W = gutter depression width, ft.
E_o = ratio of flow in the depressed section to the total flow, refer to Equation (5.3)

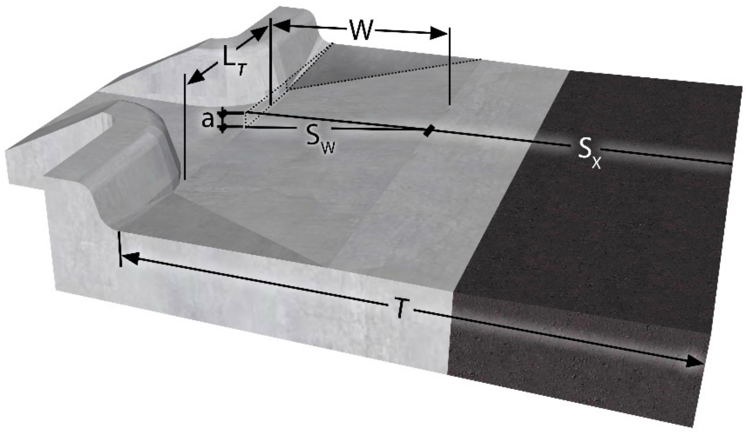


Figure 5-6 Depressed Curb Opening Cross Section

Curb-Opening Inlets in Sag

The capacity of a curb-opening inlet in a sag depends on the water depth at the curb, the curb-opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The equation for the interception capacity of a uniform curb-opening inlet operating as a weir is given by Equation (5.9). The weir equation for depressed curb-opening inlets is given by Equation (5.26).

$$Q_i = C_W L d^{1.5} \qquad (5.9)$$

$$Q_i = C_W (L + 1.8W) d^{1.5} \qquad (5.10)$$

where Q_i = flow rate, ft³/s, intercepted by the opening
C_W = weir coefficient
C_w = 3.0 for curb-openings without a depression
C_W = 2.3 for depressed curb-openings
L = length of curb opening, ft.
W = gutter depression width, ft.
d = depth of water at curb measured from the normal cross slope, ft.
$d=TS_X$ for a uniform gutter
$d=a+TS_X$ for a composite section

The weir equation is applicable to depths at the curb less than or equal to the height of the opening plus the depth of the depression.

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the height of the curb opening. The interception capacity can be computed by Equation (5.26). The depth at the inlet includes any gutter depression.

$$Q_i = C_O hL[2g(d_o)]^{0.5} \tag{5.11}$$

where Q_i = flow rate, ft^3/s, intercepted by the opening
C_O = orifice coefficient, C_O=0.67
h = height of curb-opening orifice, ft.
L = length of orifice opening, ft.
g = acceleration due to gravity, 32.2 ft/s^2
d_O = effective head on the center of the orifice throat, ft.

5.6.5 Inlet Structures

Drop structures and catch basins are used at the inlet to allow runoff to enter the sewer system. The structures also provide access for maintenance and operation. Inlet structures shall incorporate elements to capture floatable debris or sediment.

Catch basins may be connected in series before discharging to the sewer system. The minimum diameter of a catch basin receiving flow from another catch basin is 4 ft.

Inlets connecting to a combined sewer system must incorporate a water trap to prevent odors from the combined sewer from reaching the surface. When catch basins are connected in series and discharge to a combined sewer, a single water trap may be used at the catch basin structure immediately upstream of the combined sewer.

Standard Inlet Structures

Figure 5-8 illustrates standard inlet structures used along roadways.

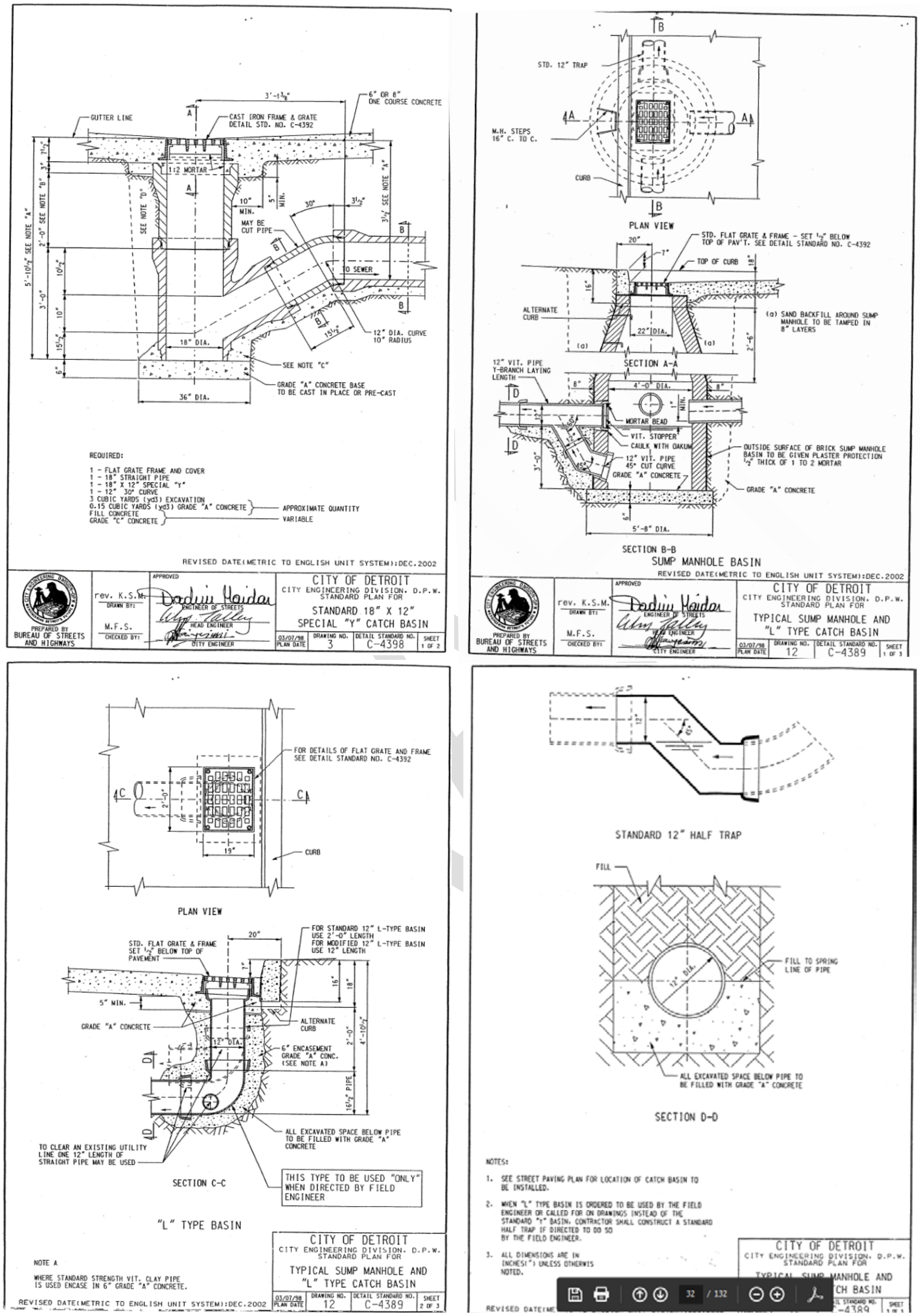


Figure 5-7 Standard Inlet Structures

Sumps

A sump may be incorporated into the design of an inlet to capture and retain sediment during low flow conditions. High flow conditions may exhibit significant washout of the captured sediment. Deeper sumps and frequent cleaning are required to minimize sediment washout. Sediment sump should be cleaned when the sediment reaches 40-50% of the sump capacity. A maintenance schedule shall be provided with the design of inlets with sumps.

The diameter and depth of the sump are recommended to be 4 times the diameter of the outlet pipe (Figure 5-8). For example, a standard 12 in diameter outlet pipe requires a 48 in diameter structure with a 48 in deep sump (measured down from the invert of the outlet pipe).

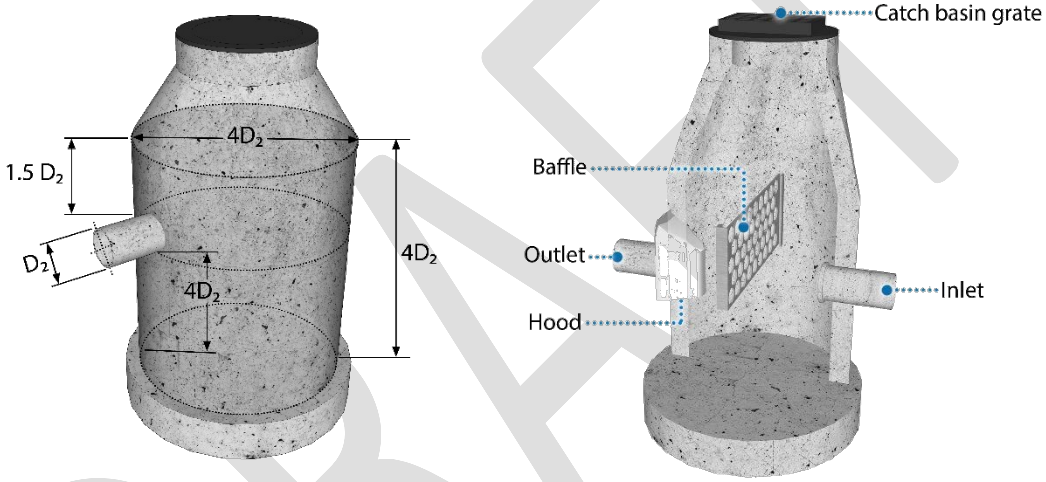


Figure 5-8 Inlet Structure with Sediment Sump

Gross Solids Removal Devices

A hood may be used over the outlet pipe to prevent floatable trash and debris from exiting the inlet structure. The hood design should incorporate an anti-siphon flow vent to prevent pollutants from being siphoned away.

Other various apparatus may be used to aid in the removal of gross solids such as screens and baffles. Care should be taken to ensure proper maintenance can be performed either around the apparatus or a plan to remove and replace the apparatus during cleaning operations. A maintenance schedule shall be provided with the design of inlets with gross solids removal devices.

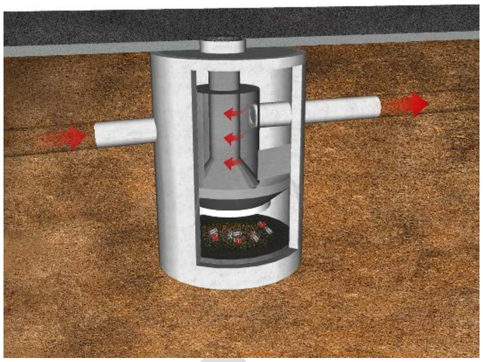


Figure 5-9 Gross Solids Removal Devices

5.7 Storm Sewers

Storm sewer systems are designed, constructed and maintained to convey stormwater runoff from: (1) one or more surface inlets to a stormwater treatment facility or outlet point; or (2) the outlet of a stormwater treatment facility to the receiving water or municipal sewer system. Storm sewers generally follow the alignment of a roadway, increasing in size as necessary to accept flow from a series of inlets. Important design parameters include return period of the design event, pipe size, pipe slope, pipe bedding, pipe cover and clearance.

The design of storm sewer systems is usually an iterative process involving the following four steps:

1. *System Layout*. Selection of inlet locations and development of a preliminary plan and profile patterns.
2. *Hydrologic Calculations*. Determination of runoff volumes and flow rates for the design storm event for the collection system.
3. *Hydraulic Calculations*. Determination of pipe sizes required to carry design flow rates and volumes.
4. *Outfall Design*. Outlet protection to prevent erosion or detention/retention to control peak discharge rates may be required because of site constraints or release-rate performance standards.

5.7.1 Design Storm

Storm sewers shall be designed for the 10 percent chance (10-year) storm. The sewer should be designed to flow full, i.e. with a hydraulic grade line at or near the top of pipe. The pipe will be allowed to surcharge in special circumstances. The surcharge elevation should be a minimum of 1 foot below grade.

In special cases, such as depressed high-volume roadways, the conveyance system shall be designed to carry the 2 percent chance (50-year) frequency event. The hydraulic grade line must be a minimum of 1 foot below grade.

Other considerations include inconvenience, hazards, and nuisances to pedestrian traffic and buildings which are located within the splash zone. These considerations should not be underestimated and, in some locations (such as commercial areas), may assume major importance.

5.7.2 Layout Requirements

Refer to Table 5-5 for minimum sewer sizes. The system should be designed for free surface flow. In difficult circumstances, it is acceptable to design for flowing full, either under pressure or not under pressure. Storm sewers shall be designed to operate under subcritical flow conditions.

Table 5-5 Minimum Sewer Size

Owned and Maintained by	Minimum Pipe Size
Publicly owned and maintained	12 in
Privately owned and maintained within the public right-of-way	12 in
Privately owned and maintained outside the public right-of way	8 in

When increasing the size of a pipe, align the inside top of the pipe (crown) where possible, resulting in a drop in the flow line. Drop manholes or other drop structures shall be used to maintain a mild pipe slope where ground slopes are steeper than critical slope.

Curved storm drains are permitted where necessary for 48-in. or larger pipe using bend sections. Smaller pipes should not be designed with curves. Long-radius bend sections are available from many suppliers and are the preferred means of changing direction in pipes 48 in. and larger. Short-radius bend sections are also available and can be used if there is not room for the long-radius bends. Deflecting the joints to obtain the necessary curvature is not desirable, except for very minor curvatures.

Storm sewers and their structures shall be kept away from building foundations or sanitary sewers as much as practicable to minimize stormwater inflow into these facilities. In instances where a proposed storm sewer will cross a sanitary sewer trench, watertight joints and trench dams shall be provided along the entire length of the proposed storm sewer from each manhole on either side of the crossing. If the storm and sanitary sewers are parallel and are within 5 feet of each other, water-tight joints and trench dams shall be installed along the entire run of the storm sewer until the distance between the storm sewer and sanitary sewer trenches exceed 5 feet.

Water-tight joints on the storm sewer and trench dams in the utility trench shall be installed at any connections to the subsurface drainage network and at the perimeter edge of all practices intended to infiltrate water into the native soil. Trench dams are used to block the flow of water at the interface of the conduit and the backfill surrounding the conduit. Trench dams may also be referred to as anti-seep collars, trench plug, and trench breaker. Trench dams may be constructed of a wide variety of materials such as concrete, clay, bentonite, metal, PVC, and HDPE.

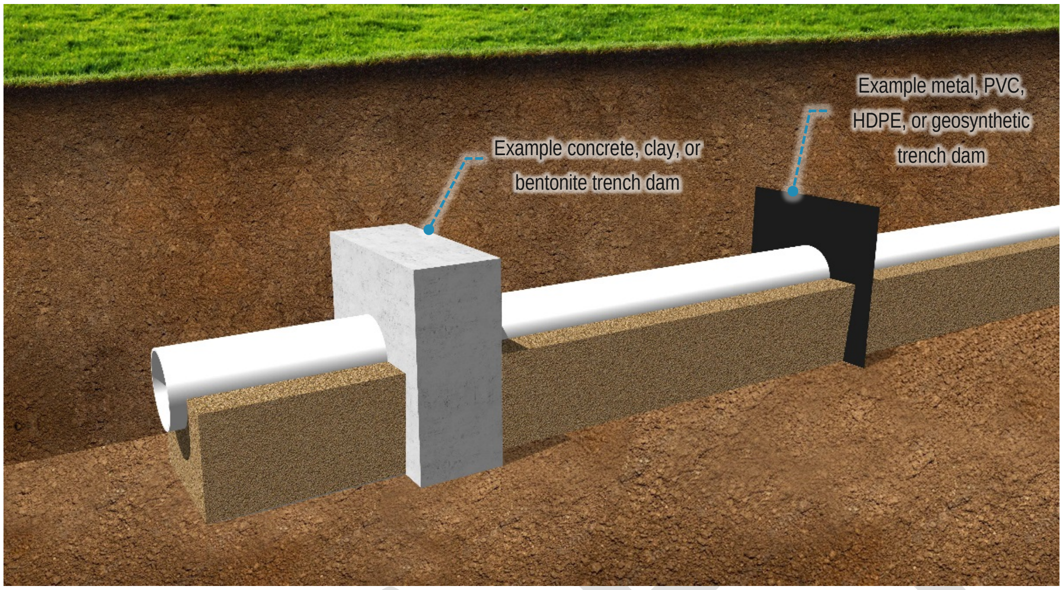


Figure 5-10 Trench dam

5.7.3 Slopes

All storm drainage systems should be designed such that velocities of flow will not be less than 3 ft/s at design flow. This criterion results in a velocity of 2 ft/s when the flow depth is 25 percent of the pipe diameter. For very flat grades, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to ensure that there is sufficient velocity (3 ft/s) in all drains to deter settling of particles. The maximum velocity should be 10 ft/s. Where velocities exceed 10 ft/s, consider adding a drop inlet to include some of the elevation change at the inlet or consider energy dissipators.

Table 5-6 Storm Sewer Slopes

Diameter (in)	Minimum Slope* 3 ft/s velocity		Maximum Slope* 10 ft/s velocity	
	Slope	Pipe Capacity (cfs)	Slope	Pipe Capacity (cfs)
8	0.75%	1.1	8.34%	3.5
12	0.44%	2.4	4.86%	7.9
15	0.32%	3.7	3.61%	12.3
18	0.25%	5.3	2.83%	17.7
21	0.21%	7.2	2.30%	24.1
24	0.17%	9.4	1.93%	31.4
30	0.13%	14.7	1.43%	49.1
36	0.10%	21.2	1.12%	70.7
42	0.08%	28.9	0.91%	96.2
48	0.07%	37.7	0.77%	126
54	0.06%	47.7	0.65%	159
60	0.05%	58.9	0.57%	196
66	0.05%	71.3	0.50%	238
72	0.04%	84.8	0.45%	283
84	0.03%	115	0.36%	385
90	0.03%	133	0.33%	442
96	0.03%	151	0.30%	503
108	0.02%	191	0.26%	636
120	0.02%	236	0.23%	785

* assumes a Manning roughness coefficient of 0.013

5.7.4 Hydraulic Grade Line

In general, if the hydraulic grade line is above the crown of a pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the hydraulic grade line is below the crown of a pipe, gravity flow calculations are appropriate. Storm sewer systems should generally be designed as gravity systems.

For storm sewers designed to operate under pressure flow conditions, inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on gravity 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.

Existence of the desired flow condition should be verified for design conditions. Storm sewer systems can alternate between pressure and gravity flow conditions from one sewer section to another.

The discharge point of the drainage system usually establishes a starting point for evaluating the condition of flow. If the discharge is submerged, as when the water level of the receiving waters is above the crown of the storm sewer, the exit loss should be added to the water level and calculations for head loss in the storm sewer system started from this point. If the hydraulic grade line is above the pipe crown at the next upstream manhole, pressure flow conditions exist; if it is below the pipe crown, then gravity flow calculations should be used at the upstream manhole.

5.7.5 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 manholes.

Exit Losses

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion, such as an endwall, the exit loss is:

$$H_o = C_o\left[\frac{V^2}{2g} - \frac{{V_d}^2}{2g}\right] \qquad (5.12)$$

where H_o = exit headloss, ft.
C_o = exit loss coefficient = 1.0
V = average outlet velocity, ft/s
V_d = channel velocity downstream of outlet, ft/s
g = acceleration due to gravity, 32.2 ft/s^2

Note that, when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For partial full flow where the pipe outlets into a channel with moving water, the exit loss may be reduced to virtually zero.

Pipe Friction Losses

The friction slope is the energy gradient in ft./ft. for that run. The friction loss is simply the energy gradient multiplied by the length of the run. Energy losses from pipe friction may be determined by rewriting Manning's equation with terms as previously defined:

$$S_f = \left[Qn/1.486AR^{2/3}\right]^2 \tag{5.13}$$

The head losses due to friction may be determined by the formula

$$H_f = S_f L \tag{5.14}$$

Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f = L\left(\frac{Qn}{0.46D^{2.67}}\right)^2 \quad for\ circular\ shapes \tag{5.15}$$

$$H_f = \frac{29n^2L}{R^{4/3}}\left(\frac{V^2}{2g}\right) \tag{5.16}$$

where H_f = headloss due to friction, ft.
S_f = slope of hydraulic grade line, ft./ft.
L = length of pipe, ft.
Q = volume flow rate, ft3/s
n = Manning's roughness coefficient, refer to Section 5.12
D = diameter of pipe, ft.
V = mean velocity, ft/s
R = hydraulic radius, ft.
g = acceleration due to gravity, 32.2 ft/s^2

Bend Losses

The bend loss coefficient for storm drainage system design is minor but can be evaluated using the formula

$$H_b = 0.0033(\Delta)\left({V_o}^2/2g\right) \tag{5.17}$$

where H_b = headloss due to a bend, ft.
Δ = angle of curvature, degrees
V = mean velocity, ft/s
g = acceleration due to gravity, 32.2 ft/s^2

Manhole Losses

The head loss encountered from one pipe to another through a manhole is commonly represented as being proportional to the velocity head at the outlet pipe. Using K_M to signify this constant of proportionality, the energy loss is

$$H_M = K_M(V^2/2g) \tag{5.18}$$

where H_M = headloss due to a manhole, ft.
K_M = manhole headloss coefficient, refer to Table 5-7
V = mean velocity, ft/s
g = acceleration due to gravity, 32.2 ft/s^2

For simple systems, an estimate or approximation of the K_M value can be used. For complex systems with complicated junctions, the K_M value should be determined using the HEC-22 (FHWA, 2013) method.

Table 5-7 Manhole Loss Coefficients

Structure Configuration	K_M
Inlet – Straight Run	0.5
Inlet – Angled Through	
90°	1.50
60°	1.25
45°	1.10
22.5°	0.70
Manhole – Straight Run	0.15
Manhole – Angled Through	
90°	1.00
60°	0.85
45°	0.75
22.5°	0.45

5.7.6 Pipe Material, Bedding and Cover

Pipe material, bedding and structural requirements shall comply with the City's Master Specifications for project contract documents. Sewers shall be designed to prevent damage from superimposed live, dead, and frost induced loads. Proper allowance for loads on the sewer shall be made because of soil and potential groundwater conditions, as well as the width and depth of trench. Where necessary, special bedding, haunching and initial backfill, concrete cradle, or other special construction shall be used to withstand anticipated superimposed loading or loss of trench wall stability.

5.7.7 Minimum Clearances

Minimum clearances for storm sewer pipe shall comply with the following criteria:

a) **Cover.** The minimum cover over the outside crown of a storm drain is 2 ft. Within roadways, a minimum of 1 foot is required between the bottom of the road base material and the outside crown of the storm drain.

b) **Utility Placement.** Storm sewer systems shall not be placed parallel to or below existing utilities, which could cause utility support problems. Electrical transmission lines or gas mains should never come into direct contact with the storm sewer.

c) **Horizontal Separation.** The minimum horizontal clearance is 3.5 ft. (10 ft. for water main) extending from each side of the storm drain and 1V:1H side slopes from the trench bottom.

d) **Vertical Crossing Separation.** Storm drains shall be constructed below watermains wherever possible. The minimum design clearance between the outside of the storm pipe and the outside of any conflicting utility shall be 1.5 ft. The crossing shall be arranged so that the storm drain joints will be equidistant and as far as possible from the joints of the crossing utility line.

   When it is impossible to obtain proper vertical separation of 1.5 ft. between the storm drain and either a watermain, sanitary, or combined sewer one of the utilities shall be placed in an approved encasement which extends 10 ft. on both sides of the crossing.

e) **Manholes.** No utility lines shall pass through any manhole or pipe. The utility line shall be offset in such a way as to provide adequate clearance between the crossings. When a sanitary line or other utility must pass through a manhole, the greatest possible clearance from the bottom of the flow channel shall be provided with a minimum of one-foot clearance.

5.7.8 Manhole

Manholes are used to provide entry to continuous underground storm drains for inspection and cleanout. Grate inlets can be used in lieu of manholes when entry to the system can be provided at the grate inlet, so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where manholes should be located are

- where two or more storm drains converge;
- where cleanouts or inspections, or both, may be required; and
- where storm drain alignment or grade changes.

Manholes may be located at other locations where drop inlets or other structures could be used. Manholes should not be in traffic lanes, bike lanes or pedestrian crosswalks.

However, where it is impossible to avoid locating a manhole in a traffic lane, ensure that it is not in the vehicular wheel path. The maximum spacing of manholes is as follows:

Table 5-8 Manhole Spacing

Size of Pipe (in)	Maximum Distance (ft.)
≤ 48	300
> 48	600

The outside diameter of all pipes entering a manhole or junction box should fit between the inside faces of the walls.

5.8 Subsurface Drainage

In certain cases, underdrains (also referred to as subsoil drains or subsurface drains) may be desired to drain excess water from the soil. A subsurface drainage network may take the form of perforated (or otherwise permeable) pipes, French drains, or collector fields. Percolation rates for groundwater may be obtained from NRCS offices, measured, or simply estimated. Collector pipe sizes and networks may then be established for the removal of that water. A maintenance schedule shall be provided with the design of subsurface drainage systems.

Underdrains are required in GSI practices with infiltration rates less than 0.5 in/hr

5.8.1 Envelope

Drain envelopes refers to the material placed on or around the underdrain. The envelope prevents excessive movement of soil particles into the drain and provides a bedding material for the drain pipe. Chapter 6, Soil, Aggregate and Water, provides information on design and selection of aggregate filters (i.e. drain envelopes).

Relative to the base soil material surrounding the drain envelope, the filter media that makes up the drain envelope should meet the following criteria (U.S. Army Corps of Engineers, 1941):

$$4 * Base\ Material\ D_{15} \leq Filter\ Media\ D_{15} \leq 4 * Base\ Material\ D_{85} \quad (5.19)$$

After calculating the D_{15} of the filter media relative to the base material, a gradation curve of the filter media may be established based on coefficient of curvature and coefficient of uniformity (ASCE, 1998).

$$C_c < \frac{{D_{30}}^2}{D_{10} * D_{60}} \tag{5.20}$$

$$C_u = \frac{D_{60}}{D_{10}} \tag{5.21}$$

where C_C = coefficient of curvature
C_U = coefficient of uniformity

Refer to Table 5-9 for recommended coefficients for drain envelope aggregate design.

Table 5-9 Drain Envelope Recommended Coefficients

Variable	Recommended Value
C_c	1 to 3 for gravels and sands
C_u	>4 for gravels >6 for sands
D_5	Not more than 5% of the filter material should pass the No. 200 sieve

The envelope material must also be large enough to not pass through the opening in the underdrain. The following criteria should be used:

$$\frac{Filter\ Media\ D_{85}}{Pipe\ Opening\ Size} \geq 1 \tag{5.22}$$

Natural aggregate material is most commonly used for the filter media. Blast furnace and steel mill slags may also be used. Geotextiles may also be used based on the properties presented in Table 5-10.

Table 5-10 Drain Envelope Geotextile Properties

Description	Recommendation	Test Method
Geotextile type	Nonwoven needle punched, knitted, or spun bonded	
Permittivity	> 1.0 sec^{-1}	
Apparent opening size	AOS ≤ D_{85} of the adjacent soil	ASTM D4751
Grab strength	> 100 pounds	ASTM D5034
Grab elongation	≤ 50%	ASTM D5034

5.8.2 Spacing

When underdrains are used to dewater a large area, the drains are commonly placed parallel to one another to provide complete dewatering. Spacing may be computed using a variety of techniques. A common steady state formula is the ellipse equation. The ellipse equation is (ASCE, 1998):

$$S = \sqrt{\frac{4K_s(m^2 - a^2)}{q}} \tag{5.23}$$

where S = drain spacing, ft.
K_S = average hydraulic conductivity, in/hr.
m = vertical distance, after drawdown of the water table above the barrier layer below the drain, ft.
a = depth of barrier layer below drains, ft.
q = drainage coefficient (steady-state surface recharge rate) in/hr.

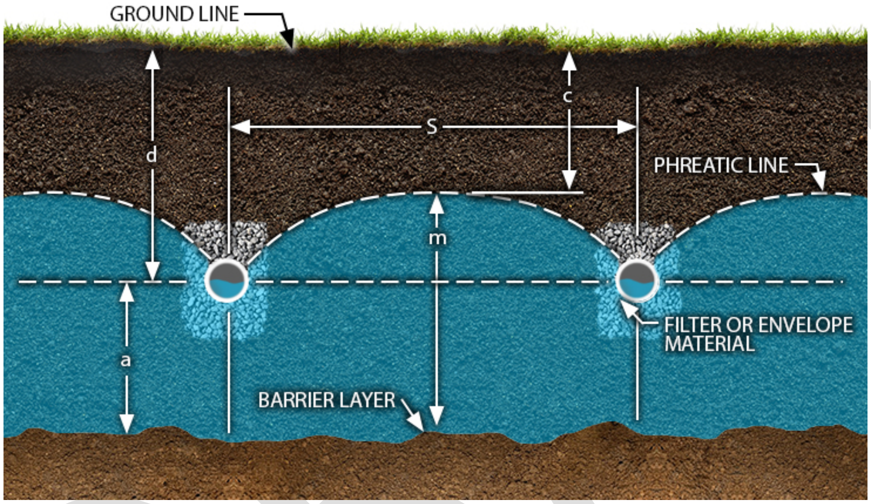


Figure 5-11 Ellipse Equation Variables

A simplified form of Darcy's Law may be used to determine the drainage coefficient in the ellipse equation.

$$q = K_s \frac{h_{max} + d}{d} \tag{5.24}$$

where q = drainage coefficient (steady-state surface recharge rate) in/hr.
K_S = average saturated hydraulic conductivity, in/hr.
h_{max} = maximum depth of ponded water above the filter bed, in
d = depth of filter media above the underdrain pipe, in

When a layer of open graded aggregate is incorporated under an entire pervious facility, the drain spacing required should be reflected in the hydraulic conductivity of the open graded aggregate.

5.8.3 Size

The flow coming to the underdrain may be estimated based on the drain coefficient (from the ellipse equation) multiplied by the surface area tributary to the underdrain. Manning's equation may then be used to calculate the pipe diameter. A minimum 4-inch diameter underdrain shall be used; 6 inches is recommended.

5.8.4 Layout

The subsurface drainage network shall consist of underdrains laid out to serve the entire desired area. Spacing should not exceed the maximum spacing calculation. Changes in alignment, vertical and horizontal, should be limited to 45 degrees to allow for inspection and maintenance. Wye fittings with removable stoppers should also be used for maintenance purposes.

Cleanouts shall be placed at the upstream end of the underdrain lines to allow for inspection and maintenance. Cleanouts are normally extended vertically up to be flush with the surface and fitted with a removable stopper.

Underdrains should discharge to a manhole or catch basin. The outlet of the drain may be fitted with an upturned elbow assembly, valve or flow reducer. Upturned elbow assemblies used to create an internal water storage zone are recommended to be constructed inside a drainage structure for easy maintenance access.

5.9 End Treatments

The purpose of an end treatment is to dissipate energy and minimize the chance of erosion at the inlet and outlet of culverts and storm sewers. Refer to Section 5.4.5 for end treatment details for culvert hydraulics.

For stormwater control measures such as bioretention and detention basins, simple end treatments should be incorporated when piping the runoff into the practice. The most common end treatment is a flared end section typically made of the same material as the pipe material. The flared end section is commonly accompanied by a riprap apron which helps to minimize erosion in the transition from a hard pipe to vegetated soil.

5.10 Outlet Channel Protection

A riprap apron is the most common outlet channel protection approach. Riprap aprons may be used for outlets up to 5 ft. diameter. The aprons are constructed of either riprap or grouted riprap placed at a zero grade for a distance based on the outlet diameter of the pipe and the flow rate.

Riprap sizing may be based on the following equation for circular pipes (Fletcher & Grace, 1972):

$$D_{50} = 0.2D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{\frac{4}{3}}\left(\frac{D}{TW}\right) \tag{5.25}$$

where D_{50} = riprap size, ft.
Q = design discharge, ft^3/s
D = culvert diameter, ft.
TW = tailwater depth, ft. Tailwater depth is to between 0.4D and 1.0D. If tailwater is unknown use 0.4D.
g = acceleration due to gravity, 32.2 ft/s^2

If the flow is supercritical in the culvert, then the culvert diameter is adjusted as:

$$D' = \frac{D + y_n}{2} \tag{5.26}$$

where D' = adjusted culvert rise, ft.
D = culvert diameter, ft.
y_n = normal (supercritical) depth in the culvert, ft.

The length and depth of the riprap apron are recommended in Table 5-11 (FHWA, 2006). The width of the apron may be determined based on a 1:3 flare.

Table 5-11 Riprap Apron Length and Depth

Class	D_{50} (in)	Apron Length	Apron Depth
1	5	4D	$3.5D_{50}$
2	6	4D	$3.3D_{50}$
3	10	5D	$2.4D_{50}$
4	14	6D	$2.2D_{50}$
5	20	7D	$2.0D_{50}$
6	22	8D	$2.0D_{50}$

D is the culvert rise or diameter

Riprap needs to be placed on a strong stable erosion-resistant base. Base soils that have a maximum of 20% fines and a minimum of 40% gravel do not need bedding. Where the natural materials do not have the necessary characteristics a bedding, or filter, material must be used. The filter layer may be either a granular material or a geotextile fabric.

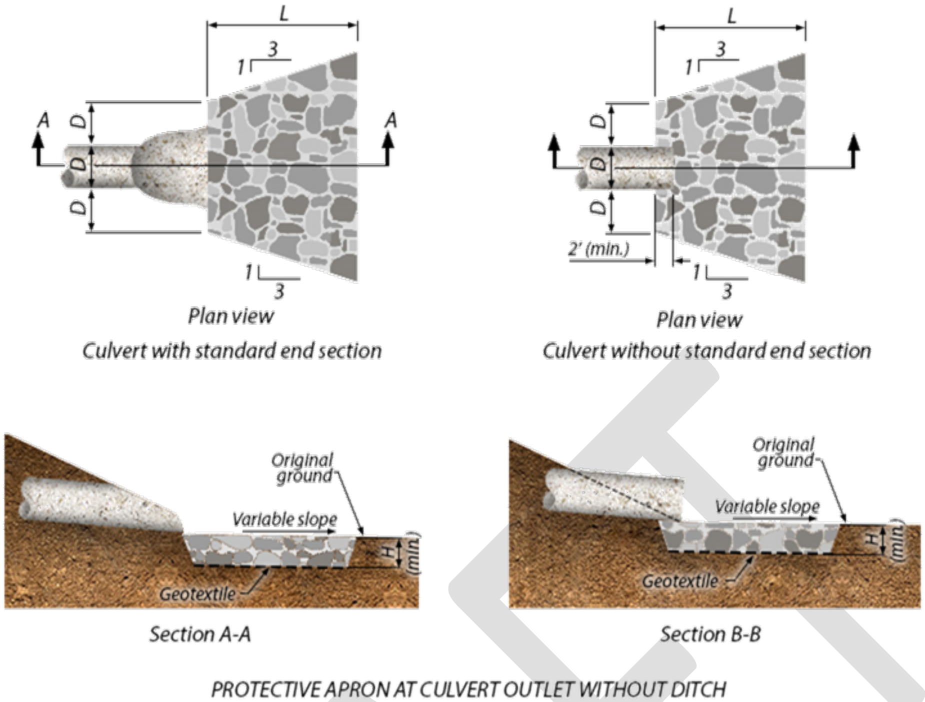


Figure 5-12 Riprap Apron

5.11 Level Spreaders

Level spreaders are used to create sheet flow conditions onto a vegetated area. The vegetated area may either be a constructed filter strip or a natural undisturbed vegetated area. Converting the runoff to sheet flow conditions spreads the water out over a large area such that runoff can be treated.

Level spreaders may be used at the edge of a large impervious surface, e.g. a parking lot, to ensure sheet flow conditions are achieved. Level spreaders may also be designed at the end of a storm pipe to convert the concentrated flow stream to sheet flow conditions.

Level Spreader Sizing

Level spreaders shall be designed for the 10-year storm. The minimum length of a level spreader is 10 ft. The maximum length is 200 ft. although maximum length of 100 ft. is recommended. Refer to Table 5-12 for sizing criteria of the weir length based on the downstream vegetation type. A bypass or diversion structure shall be provided for flows exceeding the capacity of the level spreader.

Table 5-12 Level Spreader Sizing

Downstream of the level spreader	Criteria for sizing the length of level spreader	Minimum width of vegetated filter strip receiving the flow
Engineered filter strip with turf type deep rooted grasses	10 ft. per cfs	30 ft.
Herbaceous setback/ buffer/filter strip	20 ft. per cfs	30 ft.
Wooded setback / buffer/ filter strip	50 ft. per cfs	30 ft.
Protected natural riparian buffer	50 ft. per cfs	50 ft.

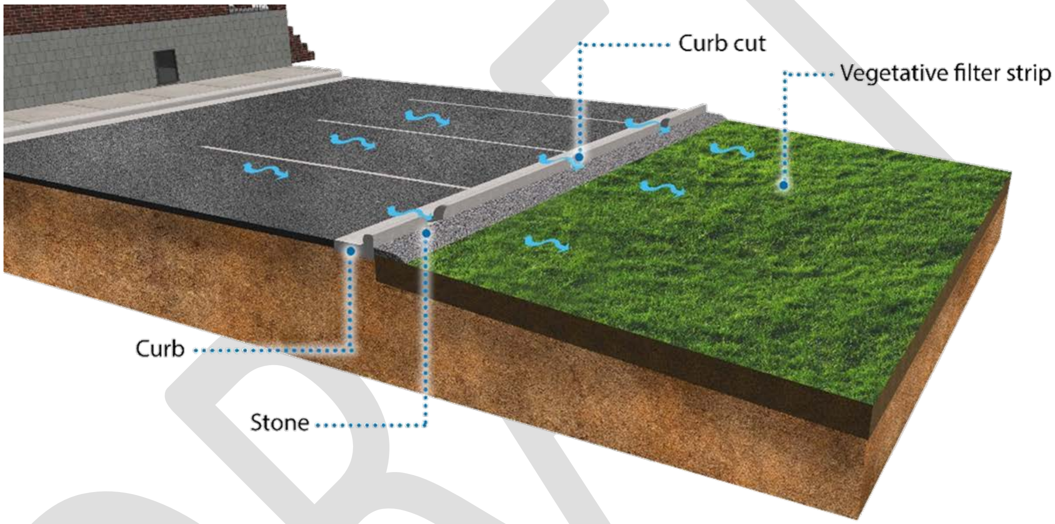


Figure 5-13 Typical Level Spreader

Flow Distribution Upstream of Level Spreader

Upstream of the level spreader a swale is traditionally provided to distribute the flow. The swale is typically a *blind* swale meaning it is a limited length with no exit. The blind swale runs parallel and adjacent to the level spreader weir. The minimum depth of the swale upstream of the level spreader is 0.5 ft. as measured down from the level lip. The depth may be increased for temporary storage capacity and to improve trapping of debris. The swale may be vegetated or constructed of a hard material such as concrete.

The angle at which the pipe upstream of the level spreader approaches the level spreader is important. Ideally the storm discharge pipe should enter the blind swale at one end and be oriented in the same direction as the blind swale. Discharging perpendicular to the blind swale can lead to short-circuiting the level spreader. If a perpendicular orientation is required, then a forebay should be included between the discharge pipe and the blind swale.

An alternative to a blind swale and weir wall is the use of a perforated pipe surrounded by aggregate. In this configuration runoff is distributed the length of the perforated pipe, exits through the perforations and flows across the vegetated filter strip. (SEMCOG, 2008)

Level Spreader Materials

The weir or lip of the level spreader is recommended to be constructed of a rigid, durable, non-erodible material such as concrete. The weir should be anchored securely below existing ground to prevent displacement. An apron of coarse aggregate shall be placed adjacent to and downstream from the rigid lip.

Underdrains may optionally be incorporated under the blind swale to provide positive drainage. If the native soils have low infiltration rates, the blind swale may not drain properly between storm events. Poor drainage will lead to problems with the vegetation and may also affect the weir wall during freeze thaw cycles.

Downstream of the Level Spreader

The longitudinal slope of the vegetated area downstream of the level spreader shall not exceed 8 percent. A longitudinal slope of 2 percent or less will maximize the filter strip performance by decreasing flow velocities.

Construction Considerations

As the name implies, level spreaders need to be constructed level. If the weir wall or lip is not constructed level the flow stream may be concentrated which can result in erosion of the vegetated filter area downstream. *Extra care needs to be taken during construction to ensure the top of the level spreader is level.*

It is recommended to use sod in the blind swale on the upstream side of the level spreader weir. This area is often hard to establish vegetation due to the incoming flows.

5.12 Manning's n Values

Manning's formula, as presented in Chow (1959), is an accepted method for performing open-channel flow capacity calculations, when uniform flow conditions represent design conditions. The selection of an appropriate resistance coefficient, known as the Manning's n value, is a key variable that requires experience and can significantly affect the results obtained. This section provides a summary of standard tables and references, which provide a consistent basis for evaluating and assigning Manning's n values. The section begins with a general discussion of basic principles for assigning n values followed by information from tabular and photographic interpretations.

5.12.1 Basic Principles

The factors presented in this section should be studied and evaluated with respect to type of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for selecting an appropriate design value of Manning's n. Consideration should also be given to the probable condition of the channel when the design event is anticipated. Values representing a freshly constructed channel are rarely appropriate as a basis for design capacity calculations.

The following basic principles should be considered when selecting the value of Manning's n:

a) **Turbulence**. Generally, retardance increases when conditions tend to induce turbulence and decreases when they reduce turbulence.

b) **Physical Roughness**. Consider the physical roughness of the bottom and sides of the channel. Fine particle soils on smooth, uniform surfaces result in relatively low values of n. Coarse materials, such as gravel or boulders, and pronounced surface irregularities cause higher values of n.

c) **Vegetation**. The value of n will be affected by the height, density, and type of vegetation. Consider the density and distribution of the vegetation along the reach and the wetted perimeter, the degree to which the vegetation occupies or blocks the cross section of flow at different depths, and the degree to which the vegetation may be bent (shingled) by flows of different depths. The n value will increase in the spring and summer as vegetation grows and foliage develops and will diminish in the fall as the vegetation becomes dormant.

d) **Cross Section**. Channel shape variations, such as abrupt changes in channel cross sections or alternating small and large cross sections, will require somewhat larger n values than normal. These variations in channel cross sections become particularly important if they cause the flow to meander from side to side.

e) **Meandering**. A significant increase in the value of n is possible if severe meandering occurs in the alignment of a channel. Meandering becomes particularly important when frequent changes in the direction of curvature occur with relatively small radii of curvature.

f) **Channel Stability.** Active channel erosion or sedimentation will tend to increase the value of n, since these processes may cause variations in the shape of a channel. Also consider the potential for future erosion or sedimentation in the channel.

g) **Obstructions.** Obstructions such as log jams or deposits of debris will increase the value of n. The level of this increase depends on the number, type, and size of obstructions.

h) **Field Observations.** Deciding on natural channel n values requires field observations and experience. Special attention is required in the field to identify floodplain vegetation and to evaluate possible roughness variations with flow depth. To be conservative, it is better to use a higher resistance for capacity calculations and a lower resistance for stability calculations.

5.12.2 Tabular Interpretations

Tables are provided below for Manning roughness coefficients for sheet flow conditions, flow along gutters, artificial channels (such as roadside swales) and for pipes.

Table 5-13 Manning n for Overland Sheet Flow

Surface Description	Manning n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipe	0.024
Cement rubble surface	0.024
Fallow (no residue)	0.05
Cultivated soils	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Range (natural)	0.13
Grass	
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Woods*	
Light underbrush	0.40
Dense underbrush	0.80

*When selecting n, consider cover to a height of about 1 inch. This is only part of the plant cover that will obstruct sheet flow.

Table 5-14 Manning n Values for Gutter Flow

Type of Gutter or Pavement	Manning n
Concrete Gutter, troweled finish	0.012
Asphalt Pavement	
Smooth texture	0.013
Rough texture	0.016
Concrete Gutter, Asphalt Pavement	
Smooth texture	0.013
Rough texture	0.015
Concrete Pavement	
Float finish	0.014
Broom finish	0.016

Table 5-15 Manning n for Artificial Channels

Lining Category	Lining Type	'n' Value for Depth of Flow Range		
		0-0.5 ft.	0.5-2.0 ft.	> 2.0 ft.
Rigid	Concrete (broom or float finish)	0.015	0.013	0.013
	Gunite	0.022	0.020	0.020
	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	0.025
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	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 (2.5-cm) d50	0.044	0.033	0.030
	2-inch (5-cm) d50	0.066	0.041	0.034
Rock Riprap	N/A	n=0.0395 (d50)1/6 d50 = Diameter of stone for which 50 percent, by weight, of the gradation is finer, in feet		

Table 5-16 Manning n for Earthen Channels

Description	Min	Normal	Max
Earth, straight and uniform			
Clean, recently completed	0.016	0.018	0.020
Clean, after weathering	0.018	0.022	0.025
Gravel, uniform section, clean	0.022	0.025	0.030
With short grass, few weeds	0.022	0.027	0.033
Earth, winding and sluggish			
No vegetation	0.023	0.025	0.030
Grass, some weeds	0.025	0.030	0.033
Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
Earth bottom and rubble sides	0.028	0.030	0.035
Stony bottom and weedy banks	0.025	0.035	0.040
Cobble bottom and clean sides	0.030	0.040	0.050
Channels not maintained, weeds and brush uncut			
Dense weeds, high as flow depth	0.050	0.080	0.120
Clean bottom, brush on sides	0.040	0.050	0.080
Same, highest stage of flow	0.045	0.070	0.110
Dense brush, high stage	0.080	0.100	0.140

Table 5-17 Manning n for Pipes

Description	Min	Normal	Max
Concrete			
Culvert, straight and free of debris	0.010	0.011	0.013
Culvert with bends, connections, and some debris	0.011	0.013	0.014
Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
Clay			
Common drainage tile	0.011	0.013	0.017
Vitrified sewer	0.011	0.014	0.017
Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
Vitrified subdrain with open joint	0.014	0.016	0.018
Brickwork			
Glazed	0.011	0.013	0.015
Lined with cement mortar	0.012	0.015	0.017
Rubble masonry, cemented	0.018	0.025	0.030
Corrugated Metal Pipe (CMP)			
2-2/3 in by ½ in corrugations annular		0.027	
2-2/3 in by ½ in corrugations helical	0.012	0.018	0.024
6 in by 1 in corrugations helical		0.025	
5 in by 1 in corrugations helical		0.026	
3 in by 1 in corrugations helical		0.028	
6 in by 2 in structural plate		0.035	
9 in by 2-1/2 in structural plate		0.037	
Spiral Rib Metal		0.013	
Polyethylene (HDPE)			
Smooth	0.009	0.012	0.015
Corrugated	0.018	0.022	0.025
Polyvinyl chloride (PVC)	0.009	0.010	0.011

5.13 Design Checklist

To ensure that all drainage conveyance systems and components have been properly designed, the following checklist shall be used. Applicable portions of this checklist can be included as part of the Post-Construction Stormwater Management Plan along with any required calculations to document design.

Treatment			
Description	**Design Requirement**	**Design Value**	**Units**
Drainage area tributary to practice/system			acres
Peak flow rate requirement	See Chapter 2 for applicable performance standards		cft/sec
Retention volume requirement	See Chapter 2 for applicable performance standards		cft

Design Flows and Volumes		
All supporting calculations and method used shall be attached		
Description	**Design Value/ Description**	**Units**
Calculation Methodology		
Minor design storm		
Major design storm		
Minor design storm volume entering practice/system		cft
Minor design storm peak flow rate entering practice/system		cft/sec
Major design storm volume entering practice/system		cft
Major design storm peak flow rate entering practice/system		cft/sec

Open Channels				
Major design storm is required to be either the 10-year or the 50-year storm				
Description	**Design Requirement**	**Recommended Values**	**Design Value/ Description**	**Units**
Hydraulic grade line elevation	Minimum of 1.5 feet below edge of road shoulder or gutter grade			ft.
Channel slope (longitudinal)	Minimum of 0.1 percent	Minimum of 0.3 percent		%
Channel shape	Trapezoidal, parabolic or triangular	Trapezoidal with minimum 2-foot bottom width or parabolic		
Maximum side slope	3H:1V	4H:1V		

Culverts				
Major design storm shall match the Open Channel design storm				
Description	**Design Requirement**	**Recommended Values**	**Design Value/ Description**	**Units**
Sizing Method	Nomographs from MDOT or FHWA, HY8, or HEC-RAS			
Type of structure and material				
Culvert diameter	Minimum of 12 inches			in
Headwater Elevation	Maximum of 1 ft below edge of road shoulder (cross-drain) or less than near edge of pavement elevation (side-drain)			ft.
Design Discharge (open channel or free fall)				
Tailwater Depth (normal depth for open channel, critical depth for free fall)				ft.
Equivalent HGL elevation (for free fall tailwater only)				
Minimum velocity (full culvert flow)	2.5 feet per second			fps
Maximum velocity (full culvert flow)	Velocities > 6 fps may require energy dissipation measures	6 fps		fps

Gutter Design			
Design storm is required to be the 10-year storm			
Description	**Design Requirement**	**Design Value/ Description**	**Units**
Roadway Classification			
Curb and gutter section type			
Design spread			ft.
Calculated maximum spread	Less than or equal to design spread based on roadway classification		ft.
Flow rate			cfs

Inlet Design				
Description	**Design Requirement**	**Recommended Values**	**Design Value/ Description**	**Units**
Inlet type				
Inlet structure diameter	Minimum of 2 feet under special circumstances, otherwise minimum of 4 feet	Minimum of 4 feet		ft.
Inlet capacity	Must intercept 100 percent of gutter flow			cfs
Length of curb-opening inlet (if used)	Must intercept 100 percent of gutter flow	Minimum of 12 inches		in
Height of curb-opening inlet (if used)				in
Maximum inlet spacing	300 feet (48" or smaller pipes) 600 feet (pipes larger than 48")			ft.
Gross solids removal device				

Inlet Locations		
Description	**Yes**	**No**
Inlets are placed on the upstream side of all bridge approaches		
Inlets are placed at all low points in gutter grade		
Inlets are placed at a maximum of 75 feet on either side of a low point or in locations that are 0.2 feet higher than the low point		
Inlets are placed on both sides of cross streets that drain toward a lower elevation roadway to prevent water from being carried across intersections		
Inlets are placed on the upstream side of driveway entrances, ramps or pedestrian crosswalks when the hydraulic analysis indicates inlets should be placed further downgrade or within features		
Inlets are placed upstream of median breaks		
Placement of inlets prevents water from sheeting across a roadway		
Inlets are placed 10 feet on either side of the intersection spring line or at the midpoint of the arc when the pavement is concrete		

Storm Sewer Design

Major design storm is required to be either the 10-year or the 50-year storm. All pipes shall be designed to flow full and operate at subcritical flow conditions.

Description	Design Requirement	Recommended Values	Design Value/ Description	Units
Minimum pipe size	8-inch (privately owned, outside of ROW) 12-inch (all other cases)			in
Maximum surcharge elevation	1 foot below grade	No surcharge		ft.
Minimum design flow velocity	3 feet per second			ft/sec
Maximum design flow velocity	10 feet per second			ft/sec
Manhole spacing	Maximum of 300 feet for pipes less than or equal to 48 inches, otherwise 600 feet			

Storm Sewer Layout		
Description	**Yes**	**No**
Pipe crowns are aligned when increasing pipe size		
Curved storm drains are only used in 48-inch or larger pipe sizes		
In addition to maximum spacing requirements, manholes are placed whenever two or more storm drains converge, where cleanouts or inspections are required, and where storm drain alignment or grade changes		
Water-tight joints and trench dams are used when parallel storm and sanitary sewers are within 5 feet of each other, at any connections to a subsurface drainage system, at the perimeter of all practices intending to infiltrate water, or from manhole to manhole when crossing a sanitary sewer trench		
Minimum horizontal distance between any storm sewer and water main is 10 feet		
Storm sewers that cross water mains have a minimum vertical clearance of 18 inches between outside diameter of pipes		
Storm sewers crossing above water mains are arranged such that joints are equidistant and as far as possible from water main joints		
All storm sewers that cannot maintain required clearance from water mains are placed in a DWSD approved encasement extending 10 feet on either side of crossings		

Subsurface Drainage System Design				
Description	**Design Requirement**	**Recommended Values**	**Design Value/ Description**	**Units**
Minimum underdrain pipe diameter	Minimum of 4 inches	Minimum of 6 inches		in
Diameter of pipe perforation				in
D85 of drain envelope aggregate	≤ diameter of pipe perforation			in
Minimum dimension of drain envelope	Minimum of 3 inches on all sides			in
Maximum spacing of parallel pipes (based on calculation)				ft.
Maximum spacing between parallel pipes (as designed)	≤ calculated maximum spacing			ft.
Subsurface drainage system capacity				cft/sec
Cleanout locations	At upstream end of all pipe runs			
Maximum change in vertical or horizontal alignment	Maximum of 45 degrees			deg

Outlet Channel Protection Design			
Description	**Design Requirement**	**Design Value/ Description**	**Units**
Outlet diameter	Maximum of 5 feet		ft.
D50 of riprap			in
Apron length			in
Apron depth			in

Level Spreader Design				
Design storm is required to be the 10-year storm				
Description	**Design Requirement**	**Recommended Values**	**Design Value/ Description**	**Units**
Length of level spreader	10 feet - 200 feet	Maximum of 100 feet		
Method of upstream flow distribution		Blind swale, weir wall		
Level spreader material	Rigid lip with downstream coarse aggregate apron	Concrete weir with downstream coarse aggregate apron		
Downstream longitudinal slope	Maximum of 8 percent	Maximum of 2 percent		%

5.14 References

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6. Soil, Aggregates and Water

One of the premises of GSI is the reduction of runoff volume principally by infiltrating the water into the subgrade. This chapter contains general information on the physical properties of soil and how water moves through the soil. The use of aggregates to provide increased storage volumes is discussed along with general information on the physical properties of the aggregates. Also discussed is the design of filters to prevent the migration of small particles into the open-graded aggregates and liners to prevent infiltration. A simple conservative computational methodology is provided for the movement of water through soil. Lastly the chapter concludes with a summary of geotechnical investigations required for the design and construction of GSI practices.

Contents

6.1 Soils

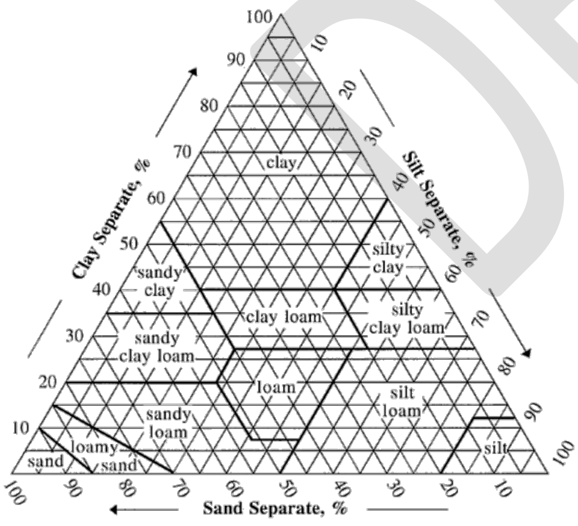
Soils are an integral component of the Earth's ecosystem and function to recycle nutrients and organic wastes, regulate gas exchange and water supplies, serve as a medium for plant growth, habitat for soil organisms, and function as an engineering medium for everything from building materials to road foundations. Given the complex interactions between physical, chemical and biological interactions that make soils so useful, designing with soils can be quite difficult. Therefore, it is essential that designers understand the complexities of soils in order to successfully design green stormwater infrastructure. The section below describes some of the pertinent properties of soils and how these properties relate to the design and function of GSI practices.

6.1.1 Soil Physical Properties

The physical properties of soils describe the characteristics of the solid particles of soil and the spaces in between. The composition of these two parts influence how well water and air move through soil, the composition of vegetation that can be supported, as well as how soil behaves in traditional engineering applications. The following physical properties are highly pertinent in GSI design and construction.

Soil Texture

Soil texture refers to the proportions of different-sized particles in a soil. The soil texture affects the soil properties and behavior. For example sand has a low capacity to hold water and a high drainage rate whereas clay holds water very well but is slow to drain. Soil texture applies only to the mineral components of soils where the particles are smaller than 2 mm in diameter.



Sand. Particles 0.05 mm to 2 mm in size. Sand feels gritty between the fingers.
Silt. Particles 0.002 mm to 0.05 in size. Silt feels smooth or silky when rubbed between the fingers.
Clay. Particles smaller than 0.002 mm. When wet, clay is sticky and can be easily molded.

A textural triangle is commonly used to show the relationship between the proportions of sand, silt and clay and the assigned textural class names (Weil & Brady, 2017).

Soil texture is used to estimate the effective porosity and hydraulic conductivity of a soil when sizing the GSI practice. Soil texture is directly used in the design of aggregate filters, geotextiles, subsurface drainage systems, soil restoration and the planting plan.

General testing methods are discussed in Methods of Soil Analysis Part 4 (Gee & Or, 2002). ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils provides a methodology to quantitatively determine the particle sizes in a soil. The distribution of particles sizes larger than 75 um is determined by sieving, while the smaller particle sizes are determined by a sedimentation process using a hydrometer. ASTM D2487 – Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) provides a standard method of classifying the soil texture based on various tests such as ASTM D422. ASTM D2488 – Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) provides a methodology to classify the soil textures based on visual examination and manual tests.

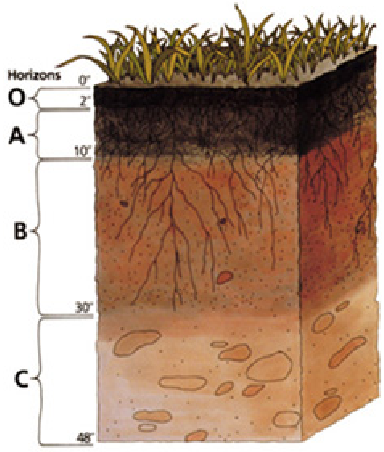
Soil Structure

In natural soils, the individual mineral particles aggregate together to form distinct structural units commonly referred to as soil aggregates or peds. The arrangement of these peds creates a network of pores through the soil that are characteristic to their location and soil composition. Soil structure determines how well water, air and organisms can move through the soil. Even soils with very high clay contents can support rapid infiltration with a well-developed soil structure.

The creation of soil structure is a lengthy process that is mainly influenced thorough physical, chemical and biological processes such as clay particle attraction, freeze-thaw cycles, penetration of roots and burrowing animals, and the production of soil glues by microorganisms. The destruction of structure through disturbance and compaction happens quite quickly and in many cases, can be impossible to recreate. Therefore, *naturally occurring soils with good structure are best protected during construction as opposed to trying to relocate them*. The processes of digging, stockpiling and spreading can easily destroy fragile peds, resulting in soils that unexpectedly fail in infiltration applications.

The Soil Profile

In natural environments, soils are not just a homogenous mix of sand silt and clay. Digging a hole into the soil reveals distinct layers that change in composition, color, texture, and structure with depth. These layers, also called horizons, form what is called the soil profile. Understanding a few basic pieces of the soil profile can provide a wealth of information about soil fertility, water holding capacity, drainage, and many other properties. In undisturbed ecosystems, the top layer of soil (O horizon) is composed of varying degrees of decaying organic matter. Organic matter from the O horizon is transported into the mineral layer below called the A horizon. The nutrient rich A horizon is commonly referred to as topsoil and contains the majority of all soil biological activity. The B and C horizons below the topsoil are much deeper and often called subsoil. Subsoil is important in determining soil strength, storing nutrients



and water for plant growth, but the conditions in these layers are not suitable for plant growth, which is why topsoil is such a valued and protected commodity.

Bulk Density

The bulk density of soil is the mass of a unit volume of dry soil. Bulk density is a measure of how tightly the soil particles are packed together, i.e. the amount of compaction. The amount of compaction affects the available pore space and rate of infiltration. Undisturbed natural forests and grasslands have a soil bulk density in the range of 0.8 to 1.1 g/cm^3. Soil bulk densities in the range of 1.4 to 1.7 g/cm^3 begin to limit the ability of plant roots to grow and penetrate the soil.

Soil compaction is important under buildings, sidewalks and roads in order to support the infrastructure. One of the principles of green stormwater infrastructure is to infiltrate rainfall and snowmelt back into the ground. In this case minimizing compaction and reducing the soil bulk density is extremely important. For more information on compaction refer to Section 6.1.2

The influence of the bulk density on the infiltration capacity is reflected in infiltration tests. A maximum bulk density may be identified in construction specification during placement of soil layers. A penetration test may be more convenient to run in the field rather than a bulk density test.

Test methods for determining the bulk density of a soil are discussed in Methods of Soil Analysis Part 4 (Grossman & Reinsch, 2002).

Porosity

Porosity refers to the volume of voids contained within a given soil sample volume and is typically expressed as a percentage. The formula for determining the porosity is the volume of voids divided by the total volume of the soil sample. As soils are compacted, bulk density increases, and the porosity of the soil decreases. Porosity may be calculated if the particle density and the dry bulk density are known.

Porosity is closely related to the void ratio, which is widely used in soil mechanics. Void ratio is defined as the volume of the voids divided by the volume of the solids.

While porosity determines the overall amount of pore space in the soil, soil pores occur in a variety of shapes and sizes that influence how water and air move through and are stored in the soil. The size, shape and interconnectedness of the pore network has a much larger effect on water movement and storage through soil than overall porosity does. The larger pores, called macropores, allow for drainage of water, movement air, and spaces roots and animals to move through the soil. Smaller pores, called micropores, are able to retain water after drainage. Much of the water used by plants is stored in the larger of these micropores.

Porosity may be estimated if the soil texture is known. Typical porosity values are provided in Table 6-1 for generalized soil texture classes. Porosity may also be estimated from soil information in the USDA Web Soil Survey or by using the SPAW software a particle size analysis test is run on a soil sample.

Test methods for determining the porosity of a soil are discussed in Methods of Soil Analysis Part 4 (Flint & Flint, 2002).

6.1.2 Soil Compaction

Soil compaction is necessary when constructing foundations, roads, parking lots, sidewalks, underground utilities and other traditional infrastructure. The compacted soil helps provide a strong stable base for the infrastructure to resist settling over time. However soil compaction results in loss of soil structure, reduced porosity, reduced infiltration and the ability of plant roots to penetrate and grow in the soil. Damaged soil structure is extremely difficult to restore and at best can only be partially restored over a short duration. Since one of the core objectives of GSI is typically to retain stormwater on site and infiltrate it, soil compaction in areas planned for stormwater management should be prevented and minimized where possible.

Compacting soil is remarkably easy. Simply digging up a natural soil with good structure and placing it back into the hole compacts it through loss of structure and subsequent settlement. Driving across soil is very damaging. Compaction of soil is more pronounced and deeper when loads (i.e. driving on) are applied on a wet soil. Soil strength decreases as water content increases and is very low when the soil is nearly saturated with water. Unfortunately, traffic across the soils is sometimes unavoidable during construction.

Ideally locations for stormwater management should be situated in areas that have not been previously compacted. A plan should be developed and executed to prevent and minimize compaction during construction. Areas planned for stormwater management and planting areas will benefit by incorporating compaction reduction techniques in the design and during construction.

Preventing and Minimizing Compaction

Prevent and minimize soil compaction by:

- Sheeting and shoring excavation to minimize the area disturbed.
- Installing exclusion fencing around areas to be undisturbed.
- Installing silt fence and other SESC measures to prevent sedimentation from occurring on the areas to be undisturbed.
- Restricting construction traffic to specific driving lanes.
- Placing mulch or wood chips on top of soil may reduce the compaction from light traffic such as pedestrians.

- Retaining existing vegetation even if it will be removed at the end of the construction.
- Providing inspection during construction to ensure the plan is being implemented.

Measuring Compaction

The most common approach to measuring compaction in the construction industry is the Proctor test (ASTM D698 Standard Proctor Test). The Proctor test defines the compaction rate as a percent of the soils maximum density at the optimum moisture level. In the field a nuclear densiometer is used for the Proctor test. A nuclear gauge only reads compaction to the depth of the probe (typically 6 to 12 inches) and is only suitable for soils with less than 5% organic content. The Proctor test is a poor way of measuring compaction of soils bioretention areas.

A penetrometer measures the resistance as a probe is slowly pushed through the soil. The device provides instant readings of the relative soil compaction. Penetration resistance is affected by the soil type and soil moisture. Root growth decreases with increasing penetration resistance, until practically stopping above 300 pounds per square inch (psi). Soil placed to grow vegetation should have a penetration resistance reading of 75 to 200 psi with a soil moisture between 5 and 15 percent.

Directly measuring the bulk density of a soil is a common method. The process involves obtaining a soil sample of known volume, drying and weighing it. Test methods for determining the bulk density of a soil are discussed in Methods of Soil Analysis Part 4 (Grossman & Reinsch, 2002).

Compaction Reduction Techniques

Breaking up compacted soils requires considerable effort. Subsoil compaction must be reduced for GSI practices intended to infiltrate stormwater. Reducing subsoil compaction is highly recommended for general landscaping areas. Various techniques may be used in an attempt to reduce compaction of soil. The two most common approaches include soil amendments and mechanical loosening.

Soil Amendment

Adding a soil amendment may provide an artificial structure to the soil and help reduce the tendency toward recompaction. Typical soil amendments include organic matter (such as a stable compost) or materials such as expanded shale or calcined clay. Amendments are typically spread over the surface before turning over the soil.

Mechanical Loosening

Loosening a compacted soil through mechanical means is the most effective technique to change the bulk density. Mechanical methods include tilling, double-spading, turning over the soil with a backhoe, subsoiling, and trench subsoiling.

Tilling, including rototilling, has a limited effectiveness in reducing compaction. Tillers typically only affect the top 6 inches of soil and can create a plow layer of compacted soil just below the tilled layer.

Double-spading is a technique gardeners use to turn over the soil by hand. It is a labor intensive method useful only in small areas. This method involves spading by hand two layers deep of soil, each layer is approximately 6 to 9 inches deep. Each layer is turned over and organic amendments are added at the top and bottom layers.

Backhoe turning is the same process as the double-spading by hand just at a bigger scale. The layer thicknesses can be increased based on the size of the backhoe bucket (buckets should be fitted with long teeth). The addition of soil amendments spread over each layer before turning is recommended. The soil surface when done will be very rough. A layer of soil amendments may be added to the surface and tilled to achieve a suitable texture for fine grading.

Subsoiling is also referred to as soil ripping. Subsoiling chisels are pulled through the soil by heavy equipment, typically fracturing the subsoil 24 to 30 inches deep. Soil amendments may be added before plowing. For best results subsoiling should be done with multiple passes in two directions, perpendicular to each other.

Trench Subsoiling uses a trenching machine to dig narrow (approximately 4 inches wide) trenches to the maximum depth possible. Trenches are dug approximately 24 to 36 inches apart and filled with a soil amendment. Soil amendments are applied across the whole surface and tilled to achieve a suitable texture for fine graded.

Mechanical loosening and the addition of soil amendments commonly results in initial soil mounding. The initial mounding will settle over time.

6.1.3 Soil Evaluation

As part of the site investigation, it is imperative to evaluate the existing soils and their suitability for GSI practices. Because both the retention and detention capacities of the practices rely so heavily on soil conditions, a variety of factors must be assessed and confirmed during the design process. The following details an initial assessment process to determine soil suitability, and soil investigations that will be necessary to confirm key soil and groundwater attributes. Additional information regarding the specific soil testing requirements that will be required during design are discussed in Section 6.7 of this chapter.

Initial Assessment

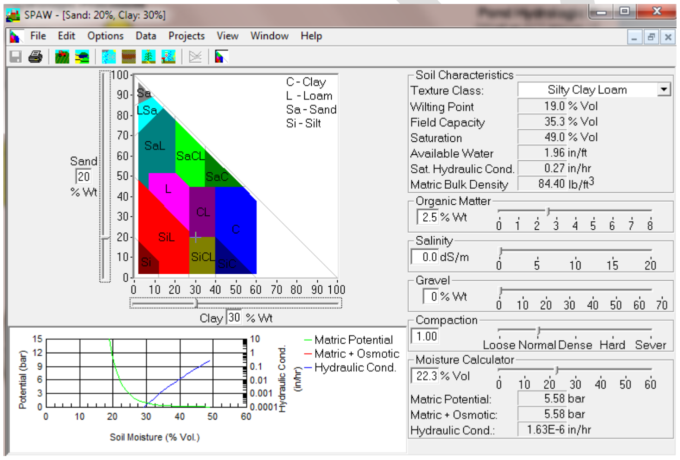
As part of the initial site investigation, the suitability of existing soils to support the necessary functions of infiltration practices needs to be evaluated. Use any available existing information regarding the expected permeability, groundwater depth, bedrock conditions and possible contamination to determine which drainage profiles would be appropriate for the site. Further field investigation to quickly assess on site soil conditions is always helpful. Digging test pits to assess soil textures, look for drainage problems and estimate infiltration rates can quickly provide information about existing soils prior to beginning design.

Soil texture can be estimated in the field through the use of the ribbon test. Taking a small handful of moist soil in the palm of your hand, roll the soil into a cigar shape with roughly ½ inch to ¾ inch diameter. Begin pressing the cigar into a flat ribbon with your thumb and forefinger. As the ribbon develops, let it extend over your forefinger until it breaks from its own weight. The length of the ribbon that can be formed before breaking indicates the texture of the soil as follows:

- Sand: soil is grainy, no ribbon is formed
- Sand loam: ½” ribbon, gritty feel
- Silty loam: less than 1” ribbon, smooth feel
- Loam: thick 1” ribbon, neither gritty nor smooth
- Silt: makes flakes instead of a ribbon
- Silty clay loam: 1-2” ribbon, thin, breaks easily, floury feel
- Sandy clay loam: 1-2” ribbon, stronger, gritty feel
- Clay loam: 1-2” ribbon, stronger, neither gritty nor smooth feel
- Sandy clay: greater than 2” ribbon, gritty feel
- Silty clay: greater than 2” ribbon, smooth feel
- Clay: long ribbon, approximately 3”

Internet Resources

.USDA Natural Resources Conservation Service (NRCS) operates the *Web Soil Survey* which is available through the internet. The survey provides soil data and information by the National Cooperative Soil Survey. Data is available throughout Detroit. Web Soil Survey does not replace the need for site specific geotechnical investigations. However the data may be used for preliminary design purposes. The Web Soil Survey webpage is available at: https://websoilsurvey.sc.egov.usda.gov



Another useful tool is the USDA Soil-Plant-Atmosphere-Water (SPAW) Field & Pond Hydrology model. Contained within the SPAW model is a Soil Water Characteristics tool. This tool provides estimations for the saturation porosity, field capacity, saturated hydraulic conductivity and other soil characteristics based on a soils sand, clay and organic matter content. The SPAW model is available at: https://hrsl.ba.ars.usda.gov/SPAW/

6.2 Soil Water

There is an intimate association between water and soil. The soil-water interactions determine the rates of water loss by leaching, surface runoff, and evapotranspiration, the balance between air and water in the soil pores, and the capacity of soil to store and provide water for plants (Weil & Brady, 2017). The characteristics and behavior of water in the soil is further discussed in this section.

6.2.1 Soil Water Content

The quantity of water present in a volume of soil constantly fluctuates as water enters and exists the soil in a response to inputs such as rainfall, removals from plant uptake and evaporation, and a number of forces that are typically quantified by soil water potentials or pressure head (ψ). Soil water content is typically reported on either a volumetric or mass basis, but can also be used to calculate a dimensionless value called the degree of saturation.

Volumetric Water Content

The volumetric water content (θ) is the volume of water associated with a given volume of dry soil. When all pores are filled with water, the soil becomes saturated (θ_s) and is at its maximum retentive capacity. As gravity and other forces pull water through the soil column, the water content decreases for a given volume and is characterized by a number of significant degrees of soil wetness that govern water storage and plant health.

Field Capacity

Field capacity (θ_{fc}) is the amount of soil moisture or water content held in the soil after excess water has drained away due to gravity. Moisture remaining is held in place due to the capillary action.

Field capacity may be estimated if the soil texture is known. Typical field capacity values are provided in Table 6-1 for generalized soil texture classes. Field capacity may also be estimated from soil information in the USDA Web Soil Survey.

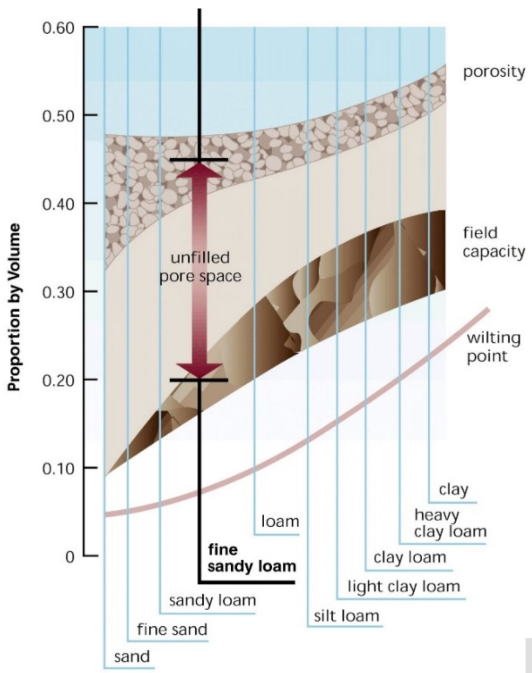
Test methods for determining field capacity are discussed in Methods of Soil Analysis Part 1 (Cassel & Nielsen, 1986).

Wilting Point

Plants may remove water from their rooting zone and the soil will continue to dry below field capacity. Plants cannot remove all of the water from the soil due to how tightly some water is held to the soil particles due to attractive forces. The term wilting point (θ_{pwp}) is used to describe the point at which plants cannot remove water from the soil fast enough to meet their needs, i.e. they begin to wilt. (Weil & Brady, 2017)

The wilting point of a soil may influence the decisions on plant selection, irrigation needs, and soil amendments.

Test methods for determining field capacity are discussed in Methods of Soil Analysis Part 1 (Cassel & Nielsen, 1986). Typical values are provided in Table 6-1.



Source: Dunne and Leopold (1978).

Fig. 2.7 -- Water-holding properties of various soils.
In Stream Corridor Restoration: Principles, Processes, and Practices, 10/98.
Interagency Stream Restoration Working Group (15 Federal Agencies of the US).

Effective Porosity

When determining the volume of water that can be stored within a soil profile, it is necessary to determine the volume of the available space. Oftentimes, this available volume is calculated using the material's void ratio or porosity. However, in soils, a portion of this void space that exists is not actually available for storage because it is already being occupied by water or trapped gasses. *Effective porosity* (n_e) is a measure of void space in a material that is readily available for the storage and drainage of water under the assumed conditions of field capacity. Effective porosity is taken as the difference between the total porosity and the field capacity and may also be referred to as the unfilled pore space.

Effective porosity is used to estimate the available space in the soil for temporary storage of stormwater runoff. Effective porosity is used directly in the design in the thickness of the bioretention soil layer.

Effective porosity is the difference between the porosity and the field capacity. Typical values are provided in Table 6-1.

6.2.2 Infiltration and Percolation

Infiltration

Infiltration is the rate at which water enters the surface of a soil. The rate of infiltration generally decreases over time as the soil becomes saturated with water. Hand calculations utilizing infiltration simplify the infiltration rate to a constant value. In these cases the infiltration rate to be used is that for fully saturated conditions.

Two commonly used equations in hydrology describe the change in infiltration with time, these are Horton's equation and the Green Ampt method. Horton's equation starts with an initial infiltration rate and decreases exponentially with time to a final infiltration rate. The Green Ampt method for infiltration is a function of the soil suction head, porosity and hydraulic conductivity. These methods are commonly used in computer models to describe infiltration.

Infiltration rates tend to be highly variable in space and time with coefficients of variation as high as 400% or more. As a result, extensive spatial and/or temporal replication of measurements is usually required for adequate hydrologic characterization of even small areas. (Reynolds, Elrick, Youngs, & Amoozegar, 2002)

Safety Factor

All infiltration rates measured in the field or estimated based on soil properties shall be divided by 2 to provide a safety factor accounting for decreased infiltration rates over time.

For design purposes assume a safety factor of 2.
Design infiltration rate is half of the measured infiltration

Percolation

Infiltration is the transition that takes place between surface water and groundwater at the soil's surface. Once water has infiltrated the soil, the continued downward movement (and to a lesser degree, lateral movement) into the soil profile is called percolation. Both saturated and unsaturated flow are involved in percolation, and water's rate of advancement into dryer soil is characterized by the movement of a boundary called the wetting front. Infiltration tests attempt to isolate and measure the vertical water movement. Percolation tests allow movement of water through both the bottom and sides of the test area. For this reason the measured rate of water level drop in a percolation test must be adjusted to account for the vertical movement only.

6.2.3 Water Movement through Soil

Darcy's Law

Darcy's Law describes the movement of water through soil. Figure 6-1 displays an experimental apparatus for the purposes of illustrating Darcy's law. Darcy's law states (Freeze & Cherry, 1979):

$$v = -K\frac{\Delta h}{\Delta l} \ or \ -K\frac{dh}{dl} \qquad (6.1)$$

where v = specific discharge, Q/A, sometimes the symbol q is used
Q = flow
A = cross sectional area
h = hydraulic head
dh/dl = hydraulic gradient, sometimes the symbol *i* is used
K = saturated hydraulic conductivity

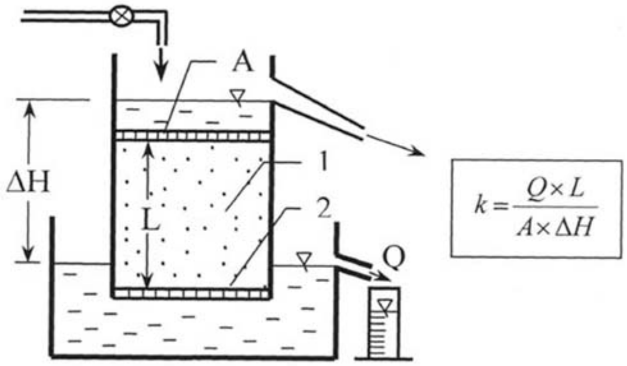


Figure 6-1 Experimental Apparatus for the Illustration of Darcy's Law

Darcy's law is an empirical equation that describes flow on a macroscopic scale, that is to say the microscopic velocities meandering between individual soil grains is ignored. Instead flow is described on a larger averaged scale. The movement of water is not limited in the downward direction, rather the flow can be in any direction based on the pressure head.

Hydraulic Conductivity

Rearranging Darcy's law equation shows that the hydraulic conductivity (K) term is the specific discharge divided by the hydraulic gradient. Hydraulic conductivity describes the ease with which a fluid moves through pore spaces. Hydraulic conductivity is depends on the intrinsic permeability of the material, the degree of saturation, and on the density and viscosity of the fluid. The units are length per time, e.g. in/hr. Hydraulic conductivity is an important property that describes how well a soil will drain.

Although the units of infiltration rate and hydraulic conductivity of soils are similar, there is a distinct difference between these two quantities. They cannot be directly related unless the hydraulic boundary conditions are known, such as hydraulic gradient and the extent of lateral flow of water.

Permeability

Permeability (k) is a measure of the ability of a porous material to allow fluids to pass through it. Equation (6.2) describes the relationship between the hydraulic conductivity and permeability.

$$K = \frac{k\rho g}{\mu} \tag{6.2}$$

where K = hydraulic conductivity
k = permeability
ρ = density of the fluid
g = gravity
μ = fluid dynamic viscosity

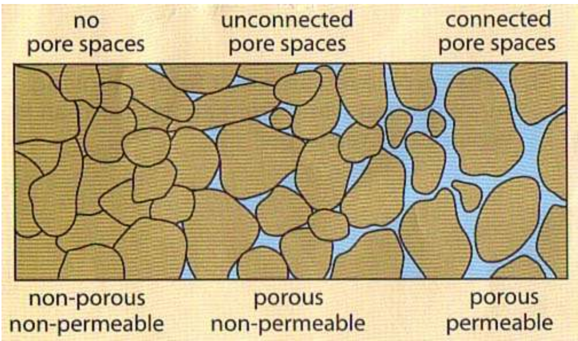


Figure 6-2 Porous vs Permeable Illustration

Saturated Soil

When all pores are filled with water and soil reaches saturation, water flow no longer causes changes in water storage, thus resulting much simplified steady-state flow conditions. Under saturated conditions, hydraulic conductivity becomes a constant term called saturated hydraulic conductivity (K_{sat}) that defines the soil's ability to transmit water under a given hydraulic gradient. This constant is the slope of the line that defines the linear relationship between the specific discharge and the hydraulic gradient (Figure 6-3). Saturated soil flow equations are further simplified in that primary forces governing the movement of water are dominated by pressure head and gravitational head. All other forces are often neglected.

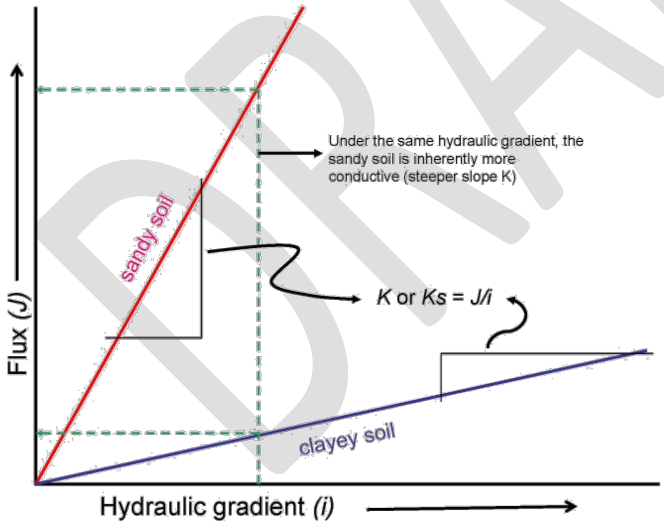


Figure 6-3 Relationship between specific discharge and hydraulic gradient

Unsaturated Soil

Soil water conditions described to this point are for saturated flow conditions. That is to say that all of the void spaces contained within the soil are assumed to be at their maximum retentive capacity (θ_s). In most real world stormwater management conditions associated with infiltrating water into the ground, flow conditions are more commonly in an unsaturated or partially saturated state. Calculating partially saturated flow can be quite difficult because as soil pores empty, the ability of water to move through the soil begins to drop drastically (Brutsaert, 2005). This means that hydraulic conductivity becomes a highly nonlinear function water content, Equation (6.3).

$$K = K(\theta) \tag{6.3}$$

where K = hydraulic conductivity
θ = volumetric water content

Typical flow equations for partially saturated soils are therefore also nonlinear and require numerical models to solve. The most popular models for variably saturated flow in one dimension are based on the Richards equation (6.4) and incorporate a variety of assumptions that must be understood before applying to specific applications.

$$\frac{\partial \theta}{\partial t} = -\frac{\partial}{\partial z}\left(K\frac{\partial H}{\partial z}\right) - \frac{\partial K}{\partial z} \tag{6.4}$$

where θ = volumetric water content
t = time
z = spatial coordinate in the vertical direction
K = hydraulic conductivity
H = water pressure head

For simplicity purposes, saturated flow conditions are commonly assumed. Unsaturated flow calculations are allowed.

6.2.4 Typical Soil Water Characteristics

This section provides typical values for the parameters previously discussed. These typical values may be used in calculations when more detailed site specific information is not known. Typical soil characteristics are shown in Table 6-1 (USEPA, 2015).

For design purposes assume the effective porosity of a bioretention soil is 20%

Table 6-1 Typical Soil Water Characteristics

Soil Texture Class	Hydraulic Conductivity K in/hr	Suction Head, Ψ, in	Porosity, ϕ, fraction	Field Capacity, θ_{fc}, fraction	Effective Porosity, fraction	Wilting Point, θ_{pwp}, fraction
Sand	4.74	1.93	0.437	0.062	0.375	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.332	0.047
Sandy Loam	0.43	4.33	0.453	0.190	0.263	0.085
Loam	0.13	3.50	0.463	0.232	0.231	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.217	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.154	0.136
Clay Loam	0.04	8.27	0.464	0.310	0.154	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.129	0.210
Sandy Clay	0.02	9.45	0.430	0.321	0.109	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.108	0.251
Clay	0.01	12.60	0.475	0.378	0.097	0.265

6.3 Simplified Soil Water Calculations

Flow calculations through a GSI practice can be complicated. Runoff soaking into a bioretention soil is typical in an unsaturated flow condition. Temporary water storage in an open-graded aggregate layer usually flows freely. Water movement into the subgrade soil may occur under various conditions. The groundwater table elevation is typically not constant but rather fluctuates seasonally.

A simple approach may be used for calculating the water entering the subgrade is based on an equivalent water depth and the infiltration rate of the in situ subgrade soil. The equivalent water depth is the effective porosity times the thickness of the media. The equivalent water depth for a bioretention practice is shown in Equation (6.5). Other similar practices use the same basic equation, just accounting for each of the different layers.

$$d_{equiv} = d_{surface} + d_{soil} * \eta_{e\ soil} + d_{aggregate} * \eta_{e\ aggregate} \qquad (6.5)$$

where d_{eqiv} = equivalent water depth, in
$d_{surface}$ = depth of the surface layer, in
d_{soil} = depth of the soil layer, in
$\eta_{e\ soil}$ = effective porosity of the soil, fraction
$d_{aggregate}$ = depth of the aggregate layer, in
$\eta_{e\ aggregate}$ = effective porosity of the aggregate layer, fraction

How much water enters the subgrade soil over a specified time period is given by Equation (6.6).

$$d_T = f_c * D \tag{6.6}$$

where d_T = total depth of water infiltrated, in
f_c = infiltration rate, in/hr
D = duration, hr

The concept of equivalent water depth can be brought into Equation (6.6) by substituting the depth of water infiltrated with the equivalent water depth. The new equation is shown in Equation (6.7).

$$d_{equiv} = f_c * D \tag{6.7}$$

Letting the infiltration rate be the measured rate for a site (including a safety factor) and the duration be the allowable infiltration duration, Equation (6.7) then provides the equivalent water depth which can be soaked into the ground for the GSI practice design.

For example, assume that the infiltration rate at a site is measured to be 0.25 in/hr. Applying the infiltration rate safety factor of 2 (refer to Section 6.2.1) the design infiltration is 0.125 in/hr. GSI practices need to be dewatered within 72 hours of the end of the rain storm; water on the surface should be infiltrated within the first 24 hours. Plugging the infiltration rate (0.125 in/hr) and the allowable dewater time (72 hr) into Equation (6.7) yields an equivalent total water depth of 9 in.

$$Total\ d_{equiv} = 0.125 \frac{in}{hr} * 72\ hr = 9\ in$$

Taking this example a step further, we can size a bioretention practice. Using the same approach for the surface dewatering duration (24 hr), the depth of water on the surface that will infiltrate within 24 hours is calculated to be 3 inches.

$$Surface\ d_{equiv} = 0.125 \frac{in}{hr} * 24\ hr = 3\ in$$

Assuming a simple bioretention system, the equivalent water depth in the soil matrix is the difference between the total equivalent depth (9 in) and the surface water depth (3 in), or 6 inches. If the effective porosity of the soil is 20%, then the depth of soil which holds an equivalent water depth of 6-inches is 30-inches.

$$Soil\ d_{equiv} = 9 - 3 = 6\ in$$

$$Soil\ depth = \frac{Soil\ d_{equiv}}{soil\ effective\ porosity} = \frac{6\ in}{0.20} = 30\ in$$

Hence our simple bioretention practice design calls for a 6-in deep surface storage zone overtop of 30 inches of bioretention soil.

6.4 Aggregates

Aggregates are a fundamental geotechnical material crucial to infrastructure construction. Aggregates are used in everything from pavements to bedding around pipes and manholes. Specific to stormwater management, aggregates are commonly used as a means to store, filter and drain the water. The type, size, shape, and strength of the aggregate materials are important characteristics to be considered during the design process.

6.4.1 Gradation

Gradation is the amount of particles of different sizes in a given sample of soil or aggregate. Gradations are determined through a sieve analysis. Refer to Table 6-2 for descriptions of common terms applied to aggregate gradations. Figure 6-4 provides illustrations of the gradation terms.

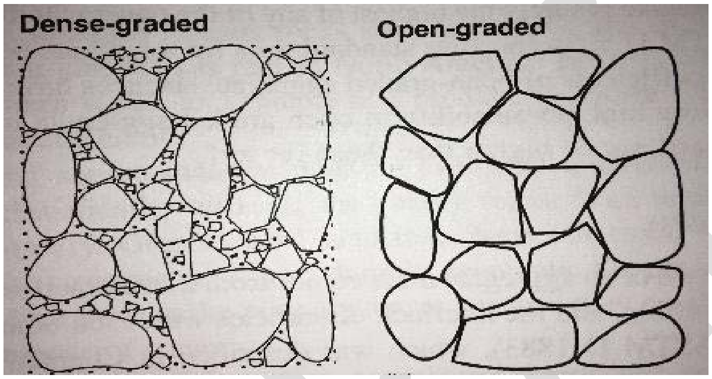


Figure 6-4 Packing of Aggregates Illustration

Provided in Table 6-3 and Table 6-4 are standard sizes of processed aggregates based on AASHTO (AASHTO, 2005) and ASTM (ASTM, 2017). Table 6-5 and Table 6-6 provides standard gradations for aggregates and granular materials. (MDOT, 2012).

Table 6-2 Gradation Descriptions

Gradation	Description
Well-graded or dense-graded	A *well-graded* or *dense-graded* material has a good representation of particle sizes over a wide range. A *dense-graded* aggregate is sized so as to contain a minimum of voids, and therefore to have the maximum weight when compacted, given a constant volume. Dense-graded materials have low porosity and hydraulic conductivity.
Coarse-graded	*Coarse-graded* materials are dense-graded with a predominance of coarse (large) particles similarly fine-graded refers to predominantly fine (small) particles.
Fine-graded	*Fine-graded* materials are dense-graded with a predominance of fine (small) particles similarly fine-graded refers to predominantly coarse (large) particles.
Poorly-graded	A *poorly-graded* material is one that there is either an excess or deficiency of certain sizes or if most of the particles are about the same size.
Open-graded	An *open-graded* aggregate is poorly-graded and is one in which a skip between the sieve gradations has been deliberately achieved so that the voids are not filled with intermediate-size particles.
Gap-graded	*Gap-graded* materials have two distinct size ranges with the intermediate sizes substantially absent.

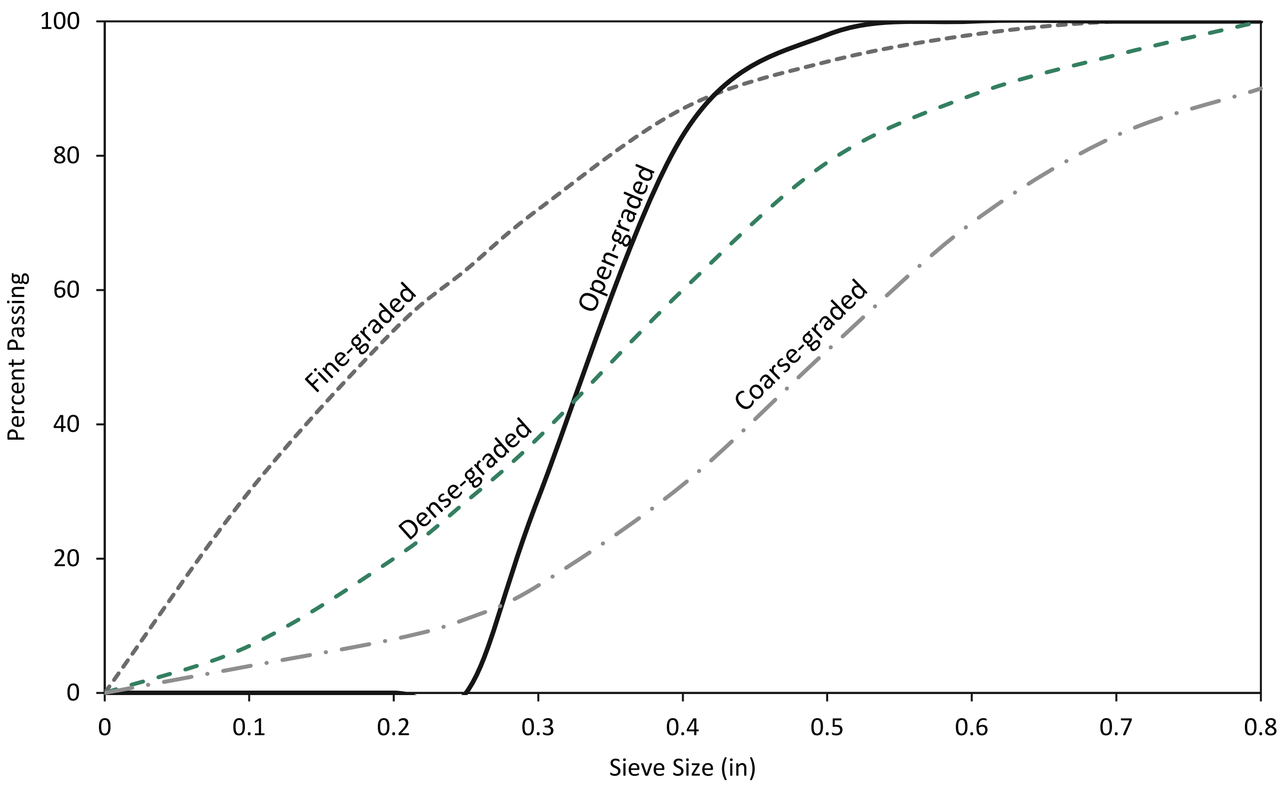


Table 6-3 ASTM and AASHTO Standard Sizes of Processed Aggregates

Size Number	Nominal Size, Square Openings	Amounts Finer Then Each Laboratory Sieve (Square Openings), Mass %														
		4 in 100 mm	3.5 in 90 mm	3 in 75 mm	2.5 in 63 mm	2 in 50 mm	1.5 in 37.5 mm	1 in 25.0 mm	3/4 in 19.0 mm	1/2 in 12.5 mm	3/8 in 9.5 mm	No. 4 4.75 mm	No. 8 2.36 mm	No. 16 1.18 mm	No. 50 300 µm	No. 100 150 µm
1	3.5 to 1.5 in 90 to 37.5 mm	100	90 to 100	-	25 to 60	-	0 to 15	-	0 to 5	-	-	-	-	-	-	-
2	2.5 to 1.5 in 63 to 37.5 mm	-	-	100	90 to 100	35 to 70	0 to 15	-	0 to 5	-	-	-	-	-	-	-
24	2.5 to 3/4 in 63 to 19.0 mm	-	-	100	90 to 100	-	25 to 60	-	0 to 10	0 to 5	-	-	-	-	-	-
3	2 to 1 in 50 to 25.0 mm	-	-	-	100	90 to 100	35 to 70	0 to 15	-	0 to 5	-	-	-	-	-	-
357	2 in to No. 4 50 to 4.75 mm	-	-	-	100	95 to 100	-	35 to 70	-	10 to 30	-	0 to 5	-	-	-	-
4	1.5 to 3/4 in 37.5 to 19.0 mm	-	-	-	-	100	90 to 100	20 to 55	0 to 15	-	0 to 5	-	-	-	-	-
467	1.5 in to No. 4 37.5 to 4.75 mm	-	-	-	-	100	95 to 100	-	35 to 70	-	10 to 30	0 to 5	-	-	-	-
5	1 to 1/2 in 25.0 to 12.5 mm	-	-	-	-	-	100	90 to 100	20 to 55	0 to 10	0 to 5	-	-	-	-	-
56	1 to 3/8 in 25.0 to 9.5 mm	-	-	-	-	-	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5				
57	1 to No. 4 25 0 to 4.75 mm	-	-	-	-	-	100	95 to 100	-	25 to 60	-	0 to 10	0 to 5	-	-	-
6	3/4 to 3/8 in 19.0 to 9.5 mm		-	-	-	-	-	100	90 to 100	20 to 55	0 to 15	0 to 5	-	-	-	-
67	3/4 in to No. 4 19.0 to 4.75 mm	-	-	-	-	-	-	100	90 to 100	-	20 to 55	0 to 10	0 to 5	-	-	-
68	3/4 in to No. 8 19.0 to 2.36 mm	-	-	-	-	-	-	100	90 to 100	-	30 to 65	5 to 25	0 to 10	0 to 5	-	-
7	1/2 in to No. 4 12.5 to 4.75 mm	-	-	-	-	-	-	-	100	90 to 100	-	40 to 70	0 to 15	0 to 5	-	-
78	1/2 in to No. 8 12.5 to 2.36 mm	-	-	-	-	-	-	-	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	-	-
8	3/8 in to No. 8 9.5 to 2.36 mm	-	-	-	-	-	-	-	-	100	85 to 100	10 to 30	0 to 10	0 to 5	-	-
89	3/8 in to No. 16 9.5 to 1.18 mm	-	-	-	-	-	-	-	-	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	-
9	No. 4 to No. 16 4.75 to 1.18 mm	-	-	-	-	-	-	-	-	-	100	85 to 100	10 to 40	0 to 10	0 to 5	-
10	No. 4 to 0 4.75 mm	-	-	-	-	-	-	-	-	-	100	85 to 100	-	-	-	10 to 30

Table 6-4 ASTM Grading Requirements for Fine Aggregate

Sieve	Percent Passing
3/8 in (9.5 mm)	100
No. 4 (4.75 mm)	95 to 100
No. 8 (2.36 mm)	80 to 100
No. 16 (1.18 µm)	50 to 85
No. 30 (600 µm)	25 to 60
No. 50 (300 µm)	5 to 30
No. 100 (150 µm)	0 to 10
No. 200 (75 µm)	0 to 3.0

Table 6-5 MDOT Standard Grading Requirements for Aggregate

Material Type	Class	Crushed Material, % min	Amounts Finer Then Each Laboratory Sieve (Square Openings), Mass %									
			2.5 in	2 in	1.5 in	1 in	3/4 in	1/2 in	3/8 in	No. 4	No. 8	No. 200
Coarse	4 AA	-	100	90 to 100	40 to 60	-	0 to 12	-	-	-	-	≤ 2.0
	6 AAA	-	-	-	100	90 to 100	60 to 85	30 to 60	-	0 to 8	-	≤ 1.0 (a)
	6 AA	-	-	-	100	95 to 100	-	30 to 60	-	0 to 8	-	≤ 1.0 (a)
	6 A	-	-	-	100	95 to 100	-	30 to 60	-	0 to 8	-	≤ 1.0 (a)
	17 A	-	-	-	-	100	90 to 100	50 to 75	-	0 to 8	-	≤ 1.0 (a)
	25 A	95	-	-	-	-	100	95 to 100	60 to 90	5 to 30	0 to 12	≤ 3.0
	26 A	-	-	-	-	-	100	95 to 100	60 to 90	5 to 30	0 to 12	≤ 3.0
	29 A	95	-	-	-	-	-	100	90 to 100	10 to 30	0 to 10	≤ 3.0
Dense Graded	21 AA	95	-	-	100	85 to 100	-	50 to 75	-	-	20 to 4	4 to 8
	21 A	25	-	-	100	85 to 100	-	50 to 75	-	-	20 to 45	4 to 8
	22 A	25	-	-	-	100	90 to 100	-	65 to 85	-	30 to 50	4 to 8
	23 A	25	-	-	-	100	-	-	60 to 85	-	25 to 60	9 to 16
Open Graded	34 R	≤ 20	-	-	-	-	-	100	90 to 100	-	0 to 5	≤ 3.0
	34 G	100	-	-	-	-	-	100	95 to 100	-	0 to 5	≤ 3.0

(a) Loss by Washing will not exceed 2.0 percent for material produced entirely by crushing rock, boulders, cobbles, slag or concrete

Table 6-6 MDOT Grading Requirements Granular Material / Fine Aggregates

Material	Sieve Analysis Total Percent Passing									
	6 in	3 in	2 in	1 in	1/2 in	3/8 in	No. 4	No. 30	No. 100	Loss by Wash Passing No. 200
Class I	-	-	100	-	45 to 85	-	20 to 85	5 to 30	-	0 to 5
Class II	-	100	-	60 to 100	-	-	50 to 100	-	0 to 30	0 to 7
Class IIA	-	100	-	60 to 100	-	-	50 to 100	-	0 to 35	0 to 10
Class IIAA	-	100	-	60 to 100	-	-	50 to 100	-	0 to 20	0 to 5
Class III	100	95 to 100	-	-	-	-	50 to 100	-	-	0 to 15
Class IIIA	-	-	-	-	-	100	50 to 100	-	0 to 30	0 to 15

Material	Sieve Analysis Total Percent Passing							
	3/8 in	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	Loss by Wash Passing No. 200
2NS	100	95 to 100	65 to 95	35 to 75	20 to 55	10 to 30	0 to 10	0 to 3.0
2SS	100	95 to 100	65 to 95	35 to 75	20 to 55	10 to 30	0 to 10	0 to 4.0
2MS	-	100	95 to 100	-	-	15 to 40	0 to 10	0 to 3.0

6.4.2 Porosity and Permeability

When constructing GSI practices, open-graded aggregates are commonly used because they provide the largest void space and hence the largest storage volume for water. The total porosity of an open-graded aggregate does not vary much with particle size. Open-graded aggregates typically have a porosity in the range of 30 to 40 percent. Open-graded aggregates also are free draining with very high permeability rates. Refer to Table 6-7 (Ferguson, 2005) for approximate permeability rates of example aggregate materials.

For design purposes assume the effective porosity of an open-graded aggregate is 40%

Table 6-7 Approximate Permeability's of Aggregate Materials

Gradation	Permeability (in/hr)
1 inch aggregate (uniform size)	25,000
½ inch aggregate (uniform size)	7,500
¼ inch aggregate (uniform size)	1,250
Coarse sand	50
Dense-graded sand and gravel	0.25

6.4.3 Shape, Strength and Durability

The shape of an aggregate is an important consideration. Rounded aggregates such as a river rock, tend to rotate and slip past each other. Rounded aggregates will roll and move when a load is applied. Crushed aggregates have angular faces which interlock with each other to resist rotating and shifting (Rollings & Rollings, 1996).

The strength of aggregates is a function of the inherent strength of the particles and particle shape, density, and gradation. Crushed aggregate particles achieve higher strengths than rounded particles because of superior interlock of the crushed particles. Aggregates must also be durable under the conditions to which they will be exposed such as wetting-drying, freezing-thawing, and impact during excavation, screening, transporting, and placement (Rollings & Rollings, 1996).

Aggregates used in stormwater management applications as bases and bedding courses under pavements or in other applications where the aggregates are under a load should be a hard, durable rock with 90% fractured faces and a Los Angeles (LA) Abrasion less than 40.

General test methods include: ASTM D5821 Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate. ASTM C535 Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine. ASTM C131

Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.

6.4.4 Crushed Concrete

Crushed concrete shall not be used in GSI applications. Water filtered through recycled crushed concrete has been shows to raise the pH of the water (Steffes, 1999). Additionally, deposits of calcium carbonate precipitate and other fines derived from the concrete material may impair drainage systems such as geotextiles and rodent screens (Snyder, 1995). High pH water can be detrimental to vegetation growth and aquatic organisms.

6.4.5 Washed

Stone aggregate bedding, base, and subbase courses should be thoroughly washed to prevent fines from clogging the subsoil interface or underdrains (Fassman & Blackbourn, 2010). Before placement, the furnished aggregate should appear free of fines and leave no substantial dust on the skin when handled. Unwashed aggregate should be replaced or washed onsite using proper construction site sediment control practices.

MDOT standard aggregates, Table 6-5, typically need to be washed thoroughly prior to using in a GSI practice. Loss by wash, i.e. passing the No. 200 sieve, should be less than 0.5%.

6.4.6 Typical Applications

Provided in Table 6-8 is a summary of typically used aggregates for various stormwater management applications. Selection of aggregates should consider locally available materials.

Table 6-8 Typical Aggregate Application

Example Application	Typical Aggregate
Joint fill material permeable interlocking concrete pavement	ASTM No. 8. Smaller sizes, e.g. No. 89 or 9, may be needed for narrower joints. MDOT 34G is equivalent in size to ASTM No. 89.
Bedding course for permeable interlocking concrete pavers	ASTM No. 8 or MDOT 34G
Reservoir/Structural layer under permeable pavements	ASTM No. 57 underlain by ASTM No. 2, 3 or 4 for deep profiles. MDOT 4AA(1), 6AA(1) or 4G may be used.
Reservoir layer under bioretention	ASTM No. 2, 3 or 4. MDOT 4AA or 6AA.
Around rigid PVC underdrain pipes	ASTM No. 57 or MDOT 6AA.

(1) Standard MDOT 4AA and 6AA are not required to be crushed. Refer to Section 6.4.3.

6.5 Filters

Filters are used to prevent the migration of fines and to maintain the structural integrated of each layer. Filter layers are inserted between two layers of materials which are not compatible. The open-graded aggregates used in GSI practices are commonly much coarser, larger, than the subgrade below the base and the surface layer above. Filters are commonly constructed of either an intermediate size aggregate or a permeable geotextile.

A filter layer over an open-graded base course layer is used to prevent small particles from the surface layer from filling in the void spaces of the base course. When the filter layer is placed over the base course it may be referred to as a *choker layer* or *transition layer*. Bioretention soil over a large open-graded aggregate base course is one example application when a choker layer may be necessary. Another example is when using an interlocking paver system, which sits on a fine aggregate setting bed, over a large open-graded aggregate base course.

Soft, plastic, fine-textured subgrade soils may flow up into the base when a load is applied particularly under very wet soil conditions. These same subgrade soils may deflect under a load. This is another case where a filter layer is needed under a base course. A geotextile filter (geogrid) is often used in this application because it can also provide structural strength. Various alternative guidelines for identifying what subgrade conditions may require some sort of geotextile at the bottom are provided in Table 6-9 (Ferguson, 2005). A professional geotechnical engineer should be consulted for determining if subgrade conditions require stabilization and designing the stabilization application if needed.

The term *plastic* or *plasticity* refers to the property of a soil to deform under a load without breaking. Plastic behavior is measured by the soil's Atterberg limits.

Table 6-9 Example Guidelines for Identifying Subgrade Conditions Requiring Geotextile

Subgrade Conditions that May Require a Geotextile	Reference
Plasticity index (PI) greater than 35, or soil expected to be saturated more than 50 percent of the time.	Burak, Rob (2002). Bases for Interlocking Concrete Pavements – The Foundation of Your Business: Part II, Interlocking Concrete Pavement Magazine February 2002, 16-18.
California Bearing Ratio (CBR) no greater than 3.	Duffy, Daniel P. (1997). Structural Reinforcement of Roadway Pavements with Geosynthetics, Erosion Control 4, 26-37.
CBR no greater than 3, or high clay or silt content, or shallow water table, or soil subject to flooding.	National Concrete Masonry Association, (1996). Concrete Grid Pavements, TEK 11-3, Herndon, Virginia: National Concrete Masonry Association.

6.5.1 Aggregate Filters

Aggregate gradation information is used to determine if a filter layer is required. Equations (6.8) thru (6.11) summarize the criteria for aggregate interfaces (Ferguson, 2005; NRCS, 1994). These equations ensure permeability and prevent migration of fines moving into or through adjacent material layers.

The D_X size is the particle diameter such that X% of the material by weight is of a smaller diameter. For example D_{15} is the particle diameter such that 15% of the material by weight is of a smaller diameter.

When comparing the aggregate size of two different layers, an uppercase "D" is used to represent the coarser or larger material and a lowercase "d" is used to represent the smaller or finer material.

When comparing the particle sizes within the same aggregate material an uppercase "D" is used.

$$\frac{D_{15}}{d_{85}} \leq 5 \tag{6.8}$$

$$\frac{D_{50}}{d_{50}} \leq 25 \tag{6.9}$$

$$\frac{D_{15}}{d_{15}} \geq 5 \tag{6.10}$$

$$C_u = \frac{D_{60}}{D_{10}} \qquad C_u \leq 6 \tag{6.11}$$

where D_{10} = particle size (diameter) at which 10 percent of the material is finer
D_{15} = particle size (diameter) for the coarser material at which 15 percent of the material is finer
D_{50} = particle size (diameter) for the coarser material at which 50 percent of the material is finer
D_{60} = particle size (diameter) at which 60 percent of the material is finer
D_{85} = particle size (diameter) for the coarser material at which 85 percent of the material is finer
d_{15} = particle size (diameter) for the finer material at which 15 percent of the material is finer
d_{50} = particle size (diameter) for the finer material at which 50 percent of the material is finer
d_{85} = particle size (diameter) for the finer material at which 85 percent of the material is finer
C_u = coefficient of uniformity

Equations (6.8) and (6.9) ensure the smaller particles don't move into the void space of the larger aggregate. Equation (6.10) evaluates the relative permeability of the two layers. In most cases the layer with the smallest permeability is desired at the top. For example in a typical permeable pavement design with interlocking pavers a large coarse aggregate layer is used under a smaller fine aggregate layer. In this case the lower coarse aggregate layer should not have the limiting permeability, hence the purpose of Equation (6.10). Equation (6.11) looks at the Coefficient of Uniformity (C_u) to prevent gap-graded and very poorly graded materials from being used.

Additional design criteria include:

- Maximum particle size is 3 inches.
- Minimum layer thickness of any one material is 4 inches.

Table 6-10 summarizes the D_{15}, D_{50}, and D_{85} particle sizes for standard aggregates (ASTM sizes are based on ASTM D488). Additional design information on aggregate filters may be found in the NRCS Part 633 National Engineer Handbook, Chapter 26: Gradation Design of Sand and Gravel Filters, 1994 (NRCS, 1994).

Table 6-10 Particle Sizes (mm) for Aggregate Filters

Standard Aggregate	D_{15} Min	D_{15} Max	D_{50} Min	D_{50} Max	D_{85} Min	D_{85} Max
ASTM #1	38.10	51.76	56.69	72.27	78.36	86.63
ASTM #10	NA	0.19	0.40	0.94	2.26	4.75
ASTM #2	38.10	43.10	45.75	53.99	56.80	62.22
ASTM #24	20.42	28.87	33.17	46.37	52.43	61.05
ASTM #3	25.40	30.22	32.88	41.21	43.99	49.49
ASTM #357	7.04	14.59	17.96	30.21	35.92	45.26
ASTM #4	19.05	23.64	24.50	30.22	33.28	37.01
ASTM #467	6.27	10.94	13.47	22.65	26.94	33.94
ASTM #5	13.29	17.21	18.21	21.55	23.08	24.88
ASTM #56	9.53	13.59	13.90	20.18	19.05	24.68
ASTM #57	5.24	8.57	10.43	16.27	19.59	23.01
ASTM #6	9.53	11.82	12.25	15.11	16.64	18.51
ASTM #67	5.13	8.00	8.82	12.82	15.12	18.13
ASTM #68	2.98	6.27	7.34	12.00	14.15	17.98
ASTM #7	4.75	6.17	7.40	10.09	11.00	12.34
ASTM #78	2.98	5.79	6.73	10.09	10.69	12.34
ASTM #8	2.81	4.98	5.79	6.88	8.21	9.53
ASTM #89	1.40	3.76	4.13	6.40	7.55	9.06
ASTM #9	1.32	2.47	2.65	3.43	3.99	4.75
MDOT 17A	5.26	6.38	8.80	12.70	14.94	18.11
MDOT 21A	0.14	0.80	3.12	12.70	16.76	25.40
MDOT 21AA	0.14	0.80	3.12	12.70	16.76	25.40
MDOT 22A	0.13	0.32	2.36	5.24	9.53	16.58
MDOT 23A	NA	0.27	1.08	6.39	9.53	17.58
MDOT 25A	2.65	5.39	5.99	8.39	8.99	11.70
MDOT 26A	2.65	5.39	5.99	8.39	8.99	11.70
MDOT 29A	2.81	4.96	5.79	6.73	8.21	9.12
MDOT 2MS	0.17	0.30	0.42	0.74	1.41	1.82

Standard Aggregate	D_{15} Min	D_{15} Max	D_{50} Min	D_{50} Max	D_{85} Min	D_{85} Max
MDOT 2NS	0.18	0.42	0.51	1.67	1.67	3.76
MDOT 2SS	0.18	0.42	0.51	1.67	1.67	3.76
MDOT 34G	2.73	2.98	4.57	5.12	7.64	8.81
MDOT 34R	2.73	2.98	4.57	5.12	7.64	8.81
MDOT 4AA	19.89	24.70	32.98	40.36	45.60	49.36
MDOT 6A	5.42	7.77	10.51	16.00	19.59	23.97
MDOT 6AA	5.42	7.77	10.51	16.00	19.59	23.97
MDOT 6AAA	5.42	7.77	10.51	16.64	19.05	24.21
MDOT Class I	0.17	2.37	1.26	14.41	12.70	34.81
MDOT Class II	0.10	0.42	0.40	4.75	2.26	50.47
MDOT Class IIA	0.09	0.42	0.33	4.75	2.14	50.47
MDOT Class IIAA	0.12	0.42	0.55	4.75	2.48	50.47
MDOT Class III	0.08	0.26	0.41	4.75	2.28	41.13
MDOT Class IIIA	0.08	0.42	0.40	4.75	2.26	7.73

6.5.2 Geotextiles

Geotextiles may be used as separators between two different materials and still provide subsurface drainage. A geotextile may be placed on the sides of the GSI practice. AASHTO M 288 provides a standard for Geotextile Specifications for Highway Applications. AASHTO M 288 is not a design guideline; it is the design professional's responsibility to choose a geotextile with site-specific variables in mind.

Table 1 of AASHTO M 288 provides three geotextile classes for strength with Class 1 representing the strongest and Class 3 the weakest (Table 6-11). The geotextile strength selection is primarily associated with the subgrade conditions, construction equipment and lift thickness, which is further elaborated on in Table 4 of the specification (Table 6-12). Class 2 is recommended as the default strength for most subsurface drainage geotextiles.

Each geotextile class is subdivided according to elongation (or stretch without breaking when placed under a load). Elongation of < 50% indicates a woven geotextile whereas an elongation of > 50% indicates a nonwoven fabric. Needle-punched nonwoven fabrics are the most often used in drainage applications.

Subsurface drainage properties are suggested in Table 6-13 based on the percent in situ soil passing the No. 200 sieve (0.075 mm) (this is based on Table 2 of AASHTO M 288). The drainage properties are for placing a geotextile against a soil and for allowing the long-term passage of water. For example, if the native in-situ soil passes 5% through a No. 200 sieve, then the geotextile selected is recommended to have a permittivity of 0.5 sec^{-1} and an apparent opening size of 0.43 mm. Woven slit film geotextiles shall not be allowed.

> Note: A professional geotechnical engineer should be consulted for designing geotextile stabilization applications. As previously discussed this is primarily when the subgrade soils are soft, plastic, and fine-textured.

Proper placement of geotextile filters is important, particularly when used with an underdrain. Never place a geotextile in a position where it will restrict flow. For example a geotextile placed between the open-graded aggregate layer and the perforated underdrain pipe restricts the flow. However, if the geotextile is placed around the outside of the open-graded aggregate layer and the adjacent soil that is to be drained, the water is constantly flowing into a medium that is more permeable, hence it is not a flow restriction. (Rollings & Rollings, 1996)

Table 6-11 Geotextile Strength Property (AASHTO M288 Table 1)

Property	Unit	Geotextile Class 1		Geotextile Class 2		Geotextile Class 3	
		Elongation <50%	Elongation ≥50%	Elongation <50%	Elongation ≥50%	Elongation <50%	Elongation ≥50%
Grab strength	N	1400	900	1100	700	800	500
	lbs	315	200	250	160	180	110
Sewn seam strength	N	1260	810	990	630	720	450
	lbs	280	180	220	140	160	100
Tear strength	N	500	350	400	250	300	180
	lbs	110	80	90	60	70	40
Puncture strength	N	2750	1925	2200	1375	1650	990
	lbs	620	430	500	310	370	220

Table 6-12 Geotextile Degree of Survivability (AASHTO M288 Table 4)

Description	Low ground pressure equipment ≤ 25 kPa (3.6 psi)	Medium ground pressure equipment 25 to 50 kPa (3.6 to 7.3 psi)	High ground pressure equipment >50kPa (>7.3 psi)
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level so that any shallow depressions and humps do not exceed 18 inches in depth or height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Low (Class 3)	Moderate (Class 2)	High (Class 1)
Subgrade has been cleared of obstacles larger than small to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 18 inches in depth or height. Larger depressions should be filled.	Moderate (Class 2)	High (Class 1)	Very High (Class 1+)
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than ± 6 inches above the subgrade. Geotextile may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed only if placing the geotextile and cover material over them will distort the finished road surface.	High (Class 1)	Very High (Class 1+)	Not recommended

Table 6-13 Subsurface Drainage Requirements (AASHTO M288 Table 2)

Property	Units	Percent in situ Soil Passing No. 200 (0.075mm)		
		<15	15 to 50	>50
Permittivity	sec^{-1}	0.5	0.2	0.1
Apparent opening size (AOS)	mm	0.43	0.25	0.22
Ultraviolet stability (retained strength)	%	50% after 500 hours of exposure		

6.6 Impermeable Liners

In situations where infiltration is not possible or not desired, the entire perimeter and bottom of the GSI practice shall be lined with an impermeable barrier. The stormwater management functions of a lined GSI practice are limited to detention and water-quality treatment, not soil infiltration. Impermeable liners may include a clay, concrete or a geomembrane. Impermeable liner systems should be designed by a licensed geotechnical engineer.

If geomembrane is used, it should be a minimum of 30 mils thick and ultraviolet resistant (ASTM D7176). A suitable geotextile fabric should be placed on the top and bottom of the membrane for puncture protection. Construction plans should specify the method for sealing the seams of the geomembrane (per manufacturer recommendations). Seams are typically heat sealed by the manufacture but can be sealed in the field following ASTM D7408 standards and all manufacturer requirements.

6.7 Geotechnical Investigations

This section details the soil investigation and infiltration testing procedures for GSI practices intended to infiltrate stormwater. Conceptual designs may be done based on USDA NRCS Web Soil Survey information along with typical soil characteristic parameters as describe in Section 6.1 and 6.2. Preliminary and final designs should be based on results of a more formal soil evaluation, with sampling and testing done in the field at the proposed sites.

Table 6-14 summarizes what geotechnical parameters are needed for infiltrating GSI practices. Additional details and discussion are provided in the subsequent sections.

Table 6-14 Summary of Geotechnical Investigations Requirements

Parameter	Purpose	How to Determine	Where and How Many Tests are Required
Soil Texture	Used to estimate the effective porosity and hydraulic conductivity of a soil when sizing the GSI practice. Soil texture is directly used in the design of aggregate filters, geotextiles, subsurface drainage systems, soil restoration and the planting plan.	Dig a test pit or soil boring. Characterize the soil profile.	**Where:** Locate tests where the proposed GSI practices will be placed. **Depth:** From the surface down to 4-ft below the bottom of the proposed GSI practice. **How Many:** 1 test site per 5,000 sq ft of GSI practices.
Infiltration and hydraulic conductivity	Used to determine the depth of water that can be infiltrated within the allowable durations for GSI practices. The surface area of the GSI practice is a directly based on the depth. Also used to design subsurface drainage systems.	Conduct an infiltration test. May be estimated based on soil texture for conceptual designs.	**Where:** Locate tests where the proposed GSI practices will be placed. **Depth:** Within 1 ft of the bottom of the proposed GSI practice. **How Many:** <u>SFR Lots</u>: 1 test site per 5,000 sq ft of GSI practices, with a minimum of 1 test per lot. <u>Non-SFR</u>: 1 test site per 5,000 sq ft of GSI practices, with a minimum of 3 test per lot.
Groundwater table	High groundwater tables may limit infiltration.	Done in conjunction with soil texture analysis	Review soil log from soil texture analysis.

6.7.1 Who Should Conduct the Investigations

It is highly recommended that all projects perform a soil evaluation and investigation by a qualified professional using the procedures detailed in the sections below. The stormwater designer is strongly encouraged to directly observe the investigation and testing process to obtain first-hand understanding of site conditions.

All projects governed by the requirements of the Post-Construction Stormwater Management Ordinance are required to perform a soil evaluation or geotechnical evaluation by a qualified professional. The professional shall perform a detailed evaluation of soils, groundwater and bedrock conditions, in addition to assessing whether or not contamination may be present. This investigation is intended to confirm soil properties and subsurface conditions that are necessary to accurately size practices and finalize design. The sections below provide detailed information on the specific geotechnical requirements for each of the testing procedures that will be necessary.

A final Geotechnical Report shall be submitted verifying the results of the required testing and characterization listed below. As part of the report, a Testing Plan must be submitted that includes layout of practices including area of practice, type of practice and depth of lowest elevation of proposed infiltration, location and type of soil test, any steep slopes or sensitive natural features such as wetlands, structural foundations, underground and overhead utilities, easements, rights-of-way, and paved surfaces.

6.7.2 Importance of GSI Practice Areas

Sites are often deemed unsuitable for GSI practices intended to infiltrate stormwater due to proposed grade changes (excessive cut or fill) or lack of suitable areas. Some sites will be constrained and unsuitable for infiltration. However, if suitable areas exist, these areas should be identified early in the design process and should *not* be subject to a building program that that precludes infiltration GSI practices. Full build-out of site areas otherwise deemed to be suitable for infiltration should not provide an exemption or waiver for adequate stormwater volume control or groundwater recharge.

6.7.3 Safety and Permits

As with all field work and testing, attention to all applicable Occupational Safety and health Administration (OSHA) regulations and local guidelines related to earthwork and excavation is required. Digging and excavation shall never be conducted without adequate notification through the Michigan One Call system (Miss Dig www.missdig.org, 811, or 1-800-482-7171). Excavations shall never be left unsecured and unmarked, and all applicable authorities shall be notified prior to any work.

Geotechnical investigations on public land require permits. A Right-Of-Entry (ROE) permit is required from the Buildings, Safety Engineering & Environmental Department (BSEED) for access to all City owned or controlled parcels and Right-Of-Way (ROW). In addition, a permit from City Engineering is required to perform work in the public ROW. Information for the permit application includes providing a detailed scope of work, site maps, drawings and certificate of insurance. The ROE permit application will be reviewed by BSEED. The ROW permit application will be reviewed by Department of Public Works (DPW), Detroit Water and Sewerage Department (DWSD), Great Lakes Water Authority (GLWA), Public Lighting Authority (PLA), Public Lighting Department (PLD), and DTE Energy Co. Additional information regarding this permit is available at: http://www.detroitmi.gov/How-Do-I/Apply-for-Permits/Right-of-Entry

Wayne County permits are required when investigations will be performed within the Wayne County ROW or on Wayne County property. More information regarding the Wayne County permit application is available at:

https://www.waynecounty.com/departments/publicservices/engineering/construction-permit.aspx

6.7.4 Soil Characterization

Overview

Soil characterization allows for estimation of the effective porosity, infiltration and hydraulic conductivity. Soil characterization provides supplementation information to infiltration testing results. Identifying limiting soil layers such as groundwater or impermeable soils, provides critical information on the feasibility and design of a GSI practice.

Soil characterization is required in areas planned for GSI practices intended to infiltrate stormwater. GSI practices that are not intended to infiltrate stormwater, e.g. water harvesting, vegetated roofs, and lined practices, are not required to characterize the in situ soil for the purposes of stormwater management. Other site development activities may require geotechnical investigations.

Soil characterization methods must be conducted along with the required soil sampling. Acceptable soil characterization testing methods include:

- Exploratory test pits, and
- Hollow-stem augered bore holes (soil borings).

Exploratory test pits allow for visual observation of the soil horizons and overall soil condition in that portion of the site. Test pits are strongly recommended over soil borings unless conditions are present that render the excavation of test pits impractical, e.g. existing structures, utilities, space constraints, and depth of the test.

Soil Characterization Requirements

General Requirements

Key requirements include:

- A minimum of 1 test pit or borehole per 5,000 square feet of GSI practice intended to infiltrate runoff.
- The location of the test pit or borehole shall correspond to the GSI practice location. Locations shall be staked and clearly labeled; stakes shall be left in the field for inspection purposes.
- The depth of the test pit or borehole shall extend a minimum of 4 feet below the bottom of the proposed GSI practice.
- A log of the soil profile shall be developed for each test pit or borehole and the soil shall be classified based on the Unified Soil Classification (USC) System and ASTM Standards D-2487 Standard Practice for Classification of Soils for Engineering Purposes and D-2488 Standard practice for Description and Identification of Soils (Visual-Manual Procedure).

- A soil sample collected within 1 vertical feet of the bottom of the proposed GSI practice shall undergo laboratory particle size analysis according to ASTM D422 Standard Test Method for Particle-Size Analysis of Soils (including a hydrometer test for particles smaller than 75 µm).
- The presence of limiting layers, groundwater presence and in situ observations of soil characteristics shall be documented.

Additional Soil Characterization Requirements for Test Pits

Additional notes and requirements for test pits include:

- Test pits are required in order to conduct double-ring infiltrometer testing.
- Appropriate sloping and benching must be provided for access and testing as necessary, in accordance with Occupational Safety and Health Administration (OSHA) Regulations.
- Upon completion of the test pit excavation and testing the pit shall be backfilled with the excavated material and compacted in place. Surface restoration in vegetated areas shall include placement of topsoil (minimum 4 in thick), an appropriate seed mix and mulch. Test pits in paved areas shall be restored with a surface acceptable to the property owner.

Additional Soil Characterization Requirements for Boreholes

Additional notes and requirements for boreholes include:

- Drilling and sampling procedures must be in accordance with ASTM D6151 Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling.
- Standard Penetration Tests (SPTs) shall be conducted in accordance with ASTM D1586 Standard Test Method for SPT and Split-Barrel Sampling of Soils. SPT shall be obtained every 2.5 feet through the top 10 feet and every 5 feet thereafter.
- Hollow-Stem augered borehole soil characterization studies must not be completed within the same hole as the infiltration testing, but must be completed no more than 25 feet away from the infiltration test locations.
- Upon completion of drilling and testing, all boreholes located in vegetated areas shall be backfilled with soil cuttings to the surface and the surface restored to conditions before drilling. Boreholes located in paved areas shall be backfilled with grout, the pavement cores placed back and the surfaces patched with cold asphalt.

6.7.5 Infiltration

Measurement Techniques

Various methodologies and devices are available to measure infiltration rates. Methods typically use either a constant or falling head of water above the surface being measured. Acceptable test methodologies for designing GSI include:

- Double-ring infiltrometer test (described later in this section).
- ASTM D3385 Standard Test Method for Infiltration Rate of Soils in Field using a Double-Ring Infiltrometer.
- Cased borehole test with soil boring (described later in this section).
- Percolation test (described later in this section).

There are differences between the test methods. The double-ring infiltrometer testing apparatus consists of two concentric metal rings that are driven into the ground and filled with water. A double-ring infiltrometer test estimates the vertical movement of water through the bottom of the test area. The outer ring helps to reduce the lateral movement of water in the soil from the inner ring. The primary differences between an infiltration test with a double-ring infiltrometer and a cased borehole are the surface area over which the infiltration test is conducted and the cased borehole test only uses a single ring (a casing installed in the borehole). Research has shown that the use of a second ring is often not effective however the accuracy of the test increases with increasing ring diameter (Reynolds, Elrick, Youngs, & Amoozegar, 2002). A percolation test allows water movement through both the bottom and sides of the test area. For this reason, the measured rate of water level drop in a percolation test must be adjusted to represent the discharge that is occurring on both the bottom and sides of the test hole.

Infiltration Test Requirements

General Requirements

Infiltration tests shall comply with the following requirements:

- Minimum number of tests required:
    - For single-family residential lots, 1 test is required per lot.
    - For non-single family residential lots a minimum of 1 test per 5,000 sq ft of GSI practice intended to infiltrate runoff and a minimum of 3 tests must be performed.
- The location of the test pit or borehole shall correspond to the GSI practice location(s). Locations shall be staked and clearly labeled; stakes shall be left in the field for inspection purposes. Testing locations should be evenly distributed.
- Each test must be accompanied by either a test pit or borehole for soil characterization (refer to Section 6.7.4).
- Tests shall be conducted in the field. The use of lab testing to establish infiltration rates is prohibited.
- Tests shall be conducted within 1 vertical ft of the bottom of the proposed GSI practice.

Additional Infiltration Test Requirements for Test Pits

Additional notes and requirements for infiltration testing in test pits include:

- Infiltration testing in test pits shall use a double-ring infiltrometer.

- Percolation tests are allowed for single family residential lots only. Non-single family residential lots are required to use a double-ring infiltrometer test.
- A maximum of two infiltration tests may be conducted per test pit.

Additional Infiltration Test Requirements for Boreholes

Additional notes and requirements for infiltration testing in boreholes include:

- Infiltration testing in boreholes shall follow the procedure for Cased Borehole Test with Soil Boring.
- Only one infiltration test is acceptable for each borehole, regardless of whether tests are proposed to be completed at different depths.
- Infiltration tests must not be completed within the same borehole as hollow-stem augered borehole soil characterization studies, but must be completed no more than 25 feet away from the soil characterization borehole locations.

Evaluation of Infiltration Testing Results

Soil infiltration rates can vary widely over short distances, even in soils that appear to be homogeneous. Measurements frequently exhibit lognormal statistical distributions, which are described using a geometric mean rather than the usual arithmetic mean (Reynolds, Elrick, Youngs, & Amoozegar, 2002). Therefore when multiple tests are conducted the geometric mean of the data must be used to determine the average infiltration rate. The geometric mean of *n* numbers is the positive n^{th} root of their product, that is to say that the geometric mean of a data set $\{a_1, a_2, \ldots a_n\}$ is given by:

$$\{a_1, a_2, \ldots a_n\} = \sqrt[n]{a_1 * a_2 * \ldots a_n} \qquad (6.12)$$

If results suggest there may be two or more distinct infiltration regimes additional investigation should be conducted to confirm this.

As discussed in Section 6.2.2 a safety factor of 2 shall be applied for design purposes.

Example of Evaluating Infiltration Test Results

Assume three infiltration tests results are collected for a site. Infiltration rates were measured to be 0.35, 0.10, and 0.08 inches per hour. The geometric mean is calculated to be 0.14 in/hr based on the formula:

$$mean = \sqrt[3]{0.35 * 0.10 * 0.08} = 0.14$$

After applying a safety factor of 2, the infiltration rate for design purposes is 0.07 in/hr.

Methodology for Double-Ring Infiltrometer Field Test

The following test method is adapted from the Low Impact Development Manual for Michigan (SEMCOG, 2008).

A double-ring infiltrometer consists of two concentric metal rings. The rings are driven into the ground and filled with water. The outer ring helps to prevent divergent flow. The drop-in water level or volume in the inner ring is used to calculate an infiltration rate. The infiltration rate is the amount of water per surface area and time unit which penetrates the soils. The diameter of the inner ring should be approximately 50-70 percent of the diameter of the outer ring, with a minimum inner ring size of four inches. Double-ring infiltrometer testing equipment designed specifically for that purpose may be purchased. However, field testing for GSI design may also be conducted with readily available materials.

Equipment for double-ring infiltrometer test:

Two concentric cylinder rings six inches or greater in height. Inner ring diameter equal to 50-70 percent of outer ring diameter (i.e., an eight-inch ring and a 12-inch ring). The diameter of the inner ring must be no less than 6 in. Material typically available at a hardware store may be acceptable.

- Water supply,
- Stopwatch or timer,
- Ruler or metal measuring tape,
- Flat wooden board for driving cylinders uniformly into soil,
- Rubber mallet, and
- Log sheets for recording data.

Procedure for double-ring infiltrometer test

1. Prepare level testing area.
2. Place outer ring in place; place flat board on ring and drive ring into soil to a minimum depth of two inches.
3. Place inner ring in center of outer ring; place flat board on ring and drive ring into soil a minimum of two inches. The bottom rim of both rings should be at the same level.
4. The test area should be presoaked immediately prior to testing. Fill both rings with water to water level indicator mark or rim at 30-minute intervals for one hour. The minimum water depth should be four inches. The drop in the water level during the last 30 minutes of the presoaking period should be applied to the following standard to determine the time interval between readings:
    a. If water level drop is two inches or more, use 10-minute measurement intervals.
    b. If water level drop is less than two inches, use 30-minute measurement intervals.
5. Obtain a reading of the drop in water level in the center ring at appropriate time intervals. After each reading, refill both rings to water level indicator mark or rim. Measurement to the water level in the center ring should be made from a fixed reference point and should continue at the interval determined until a minimum of eight readings are completed or until a stabilized rate of drop is

obtained, whichever occurs first. A stabilized rate of drop means a difference of ¼ inch or less of drop between the highest and lowest readings of four consecutive readings.

6. The drop that occurs in the center ring during the final period or the average stabilized rate, expressed as inches per hour, should represent the infiltration rate for that test location.

Methodology for Cased Borehole Test with Soil Boring

The following test method is adapted from the Philadelphia Water Stormwater Management Guidance Manual (Philadelphia Water, 2015).

The borehole infiltration method is based on a slightly modified ASTM D6391 standard. This test method may be used for compacted fills or natural deposits that have a mean hydraulic conductivity less than or equal to 10^{-3} cm/s (1.4 in/hr). The modified procedure avoids the use of a bentonite paste at the tip of the casing and a bentonite seal within the annular space between the casing and the surrounding soils. The use of bentonite can absorb moisture from the surrounding soils before swelling and hardening. As a result, the test results may not be accurate.

1. Advance a borehole to the depth of the proposed infiltration interface depth using the Hollow-Stem Auger Method (ASTM D6151). The augered hole diameter must be at least two inches larger than the outer diameter of the inner casing. The inner casing will consist of a PVC pipe with minimum inner diameter of four inches and a smooth, square bottom.
2. Push the inner casing within the auger hollow stem to the infiltration interface and firmly set into the bottom of the borehole. Use a borehole plane to scarify the soil surface at the bottom of the casing and remove any remaining loose soil. Measure the depth from the top of casing to the bottom of the hole to the nearest 0.01 feet.
3. Remove the augers.
4. Place two inches of fine gravel or coarse sand in the bottom of the borehole to prevent scour during filling of the casing. Be sure to place gravel or sand uniformly to obtain an even depth within the hole. Re-measure the depth from the top of casing to the gravel or sand surface to the nearest 0.01 feet.
5. Presoak test holes immediately prior to testing to simulate saturated conditions. Fill casing with water at a very low rate so as not to disturb the bottom sediments. Place water to a depth of at least six inches above the bottom and readjust every 30 minutes for one hour. A constant head can be applied and maintained at the top of the casing as an alternate method. The drop in the water level during the last 30 minutes of the presoaking period must be applied to the following standard to determine the time interval between readings:
    a. If water level drop is two inches or more, use ten-minute measurement intervals.
    b. If water level drop is less than two inches, use 30-minute measurement intervals.
6. After the presoaking, the water level is measured, using an approved method per the ASTM standard, where the water level remains at least 12 inches above

the bottom of the hole. All water added must be recorded as a volume along with the time of addition.

7. Measurements of water level must be made from the top of casing and must continue at the interval determined until a minimum of eight readings are completed or until a stabilized rate of drop is obtained, whichever occurs first. A stabilized rate of drop means a difference of 0.25 inch or less of drop between the highest and lowest readings of four consecutive readings.
8. Upon completion, remove casing and backfill hole with cuttings. If testing is conducted in vegetated areas, return the surface to its previous state. If testing is completed in paved areas, plug the hole with a bentonite plug, and seal the surface with concrete or asphalt.
9. Calculate the infiltration rate as described in ASTM D6391.

Methodology for Percolation Test

The following test method is adapted from the Low Impact Development Manual for Michigan (SEMCOG, 2008).

Equipment for percolation test

- Post hole digger or auger,
- Water supply,
- Stopwatch or timer,
- Ruler or metal measuring tape,
- Log sheets for recording data,
- Knife blade or sharp-pointed instrument (for soil scarification),
- Course sand or fine gravel, and
- Object for fixed-reference point during measurement (nail, toothpick, etc.).

Procedure for percolation test

This percolation test methodology is based largely on the criteria for onsite sewage investigation of soils.

1. Prepare level testing area.
2. Prepare hole having a uniform diameter of 6-10 inches and a depth of 8-12 inches. The bottom and sides of the hole should be scarified with a knife blade or sharp-pointed instrument to completely remove any smeared soil surfaces and to provide a natural soil interface into which water may percolate. Loose material should be removed from the hole.
3. (Optional) Two inches of coarse sand or fine gravel may be placed in the bottom of the hole to protect the soil from scouring and clogging of the pores.
4. Test holes should be presoaked immediately prior to testing. Water should be placed in the hole to a minimum depth of six inches over the bottom and readjusted every 30 minutes for one hour.
5. The drop in the water level during the last 30 minutes of the final presoaking period should be applied to the following standard to determine the time interval between readings for each percolation hole:

   a. If water remains in the hole, the interval for readings during the percolation test should be 30 minutes.
   b. If no water remains in the hole, the interval for readings during the percolation test may be reduced to 10 minutes.

6. After the final presoaking period, water in the hole should again be adjusted to a minimum depth of six inches and readjusted when necessary after each reading. A nail or marker should be placed at a fixed reference point to indicate the water refill level. The water level depth and hole diameter should be recorded.
7. Measurement to the water level in the individual percolation holes should be made from a fixed reference point and should continue at the interval determined from the previous step for each individual percolation hole until a minimum of eight readings are completed or until a stabilized rate of drop is obtained, whichever occurs first. A stabilized rate of drop means a difference of ¼ inch or less of drop between the highest and lowest readings of four consecutive readings.
8. The drop that occurs in the percolation hole during the final period, expressed as inches per hour, should represent the percolation rate for that test location.

Convert percolation tests results to infiltration rates

The average measured rate must be adjusted to account for the discharge of water from both the sides and bottom of the hole and to develop a representative infiltration rate. The measured percolation rate should be adjusted to represent an infiltration rate based on Equation (6.13) and (6.14).

$$f = \frac{PR}{R_f} \tag{6.13}$$

where f = Infiltration rate, in/hr
PR = percolation rate
R_f = reduction factor

$$R_f = \frac{2d_i - \Delta d}{D} + 1 \tag{6.14}$$

where d_i = initial water depth, in
Δd = average/final water level drop, in
D = diameter of the percolation hole, in

In most cases, the reduction factor varies from about two to four depending on the percolation hole dimensions and water level drop – wider and shallower tests have lower reduction factors because proportionately less water exfiltrates through the sides.

The area reduction factor accounts for the exfiltration occurring through the sides of percolation hole. It assumes that the percolation rate is affected by the depth of water in the hole and that the percolating surface of the hole is in uniform soil. If there are *significant problems with either of these assumptions then other adjustments may be necessary.*

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7. Detention Practices

Urban development results in increased runoff volumes and flowrates, which may cause frequent flooding. Detention practices are stormwater management practices designed to limit adverse downstream effects of urban storm runoff. This chapter discusses three basic types of detention practices including a traditional detention practice, an extended dry detention practice, and an extended wet detention practice. Also discussed includes placing the detention practice underground, on the surface of a parking lot, and constructing a practice for water harvesting and reuse applications. The various components that make up a detention practice are outlined followed by design standards and an overview of the calculation and sizing methodologies.

Contents

7.1 Introduction

Detention practices are small impoundments of stormwater runoff. Detention practices are one method used primarily to meet peak flow control requirements and prevent local flooding. Their design may also include features to control water quality. A detention practice should be the final element in the stormwater management design sequence for a site. This section is intended to focus on local and regional impoundments to manage, treat, and attenuate stormwater runoff thus reducing the impact on downstream areas.

There are many variations of detention practices. For the purposes of this design manual the term *detention* refers to an impoundment of stormwater runoff for the sole purpose of limiting the peak flow rate downstream of the practice. Hence a traditional detention practice is intended to limit the peak flow of stormwater runoff. A traditional detention practice can be thought of as a simple depressed storage area with a flow control structure at the downstream end. The term *extended* detention refers to a practice intended to manage the peak flow *and* improve water quality. Extended detention practices may be constructed such that they are dry in between rainfall events or with a permanent pool of water. Dry versus wet practices provide various levels of water quality improvements and different site amenities. The following types of detention practices are discussed in this chapter.

- *Detention* practices are impoundments of stormwater runoff intended to prevent downstream flooding by attenuating peak discharge rates. Reducing the flow rates in the combined sewer system leads to fewer combined sewer overflows. Detention practices are common because of their comparatively low cost, few design limitations, ability to serve large and small watersheds, and potential to be incorporated into other uses (e.g., recreational areas). They are typically dewatered within 24 hours after a rain event has ended and are intended to be dry in between rain events. Traditional detention practices are not intended to provide water quality improvements.

Figure 7-1 Traditional Detention Practice

- *Extended Dry Detention* practices provide an extended detention time for water quality improvements. These practices include a forebay for sediment pretreatment, have an elongated shape, and may include a micropool. The outlets are designed such that stormwater runoff is detained for a period of time, typically 24 hours to 72 hours. The temporary storage allows sediment to settle out; overall, however, extended dry detention practices are minimally effective in removing pollutants compared to other stormwater control measures.
- *Extended Wet Detention* practices add a permanent pool of water to an extended dry detention practice. The permanent pool of water provides improved water quality benefits. Storage is provided above the level of the permanent pool. Sometimes these practices are referred to as wet ponds or retention.
- *Subsurface Detention* practices store the water in an underground tank or chamber. The system may be prefabricated or constructed on site and may allow infiltration through the bottom. This method is most applicable where space is constrained.
- *Parking Lot Detention* allows shallow ponding within paved portions of a parking lot. Parking lot storage is a convenient method where impervious parking lots are planned.
- *Practices Used for Harvesting and Reuse* may also serve to provide detention for a site. These permanent ponds are multi-purpose, providing stormwater retention, sedimentation, and storage for later use. In this way, stormwater harvest and reuse systems can be part of a treatment train approach.

Detention can share the same footprint as other types of GSI practices. For example a bioretention practice may be constructed in the bottom of a detention practice. The primary concern of such a design is establishing vegetation that will thrive in a wide variety of inundation conditions. Detention may also be combined with a porous pavement system. In this case, additional detention storage is provided under the porous pavement either by increasing the depth of the open graded aggregate or providing some subsurface detention.

Practices should be located where topography allows for maximum runoff storage at minimum excavation or embankment construction costs. When locating a detention practice consider the location and use of other land use features, such as planned open spaces and recreational areas, and if a multi-use objective can be safely achieved with the practice.

7.2 Components

Detention practices are constructed with several different components depending on site conditions and the desired results. Refer to Figure 7-2 for an example layout and the corresponding components.

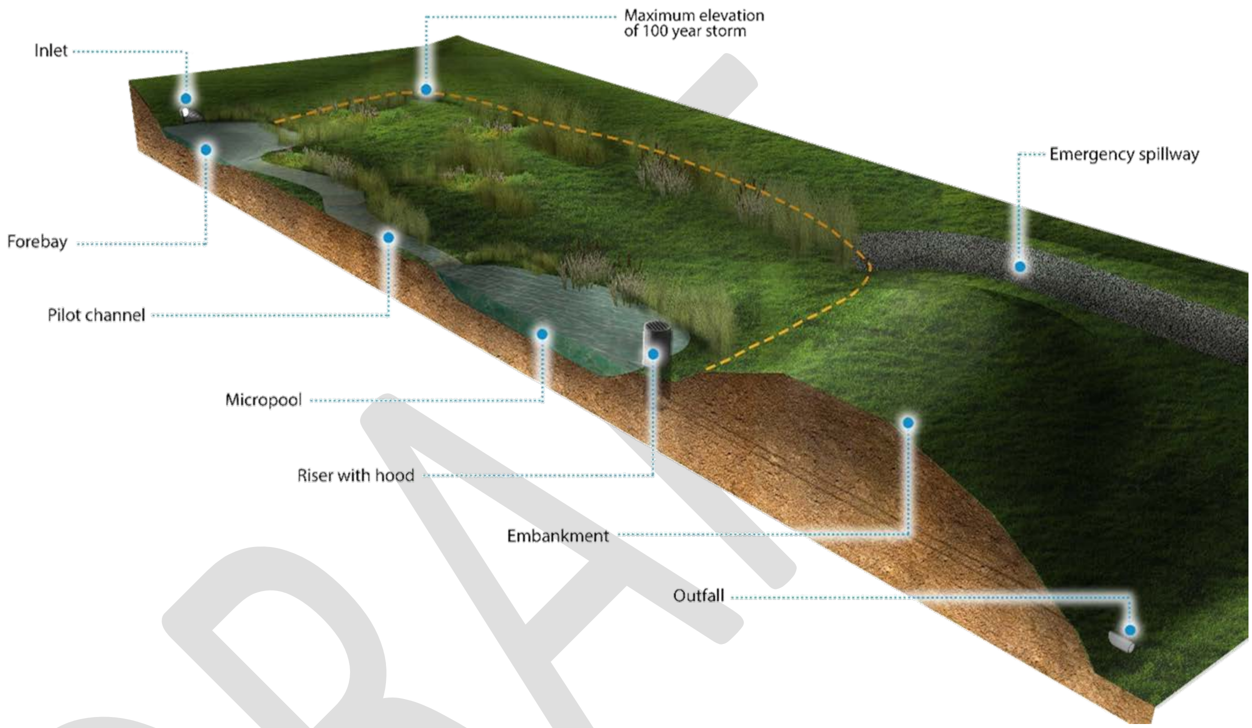

Figure 7-2 Extended Dry Detention with Micropool

Inlet

Water enters the detention practice through the inlet. Inlets may be pipes or channels and need to be stabilized to prevent scour and erosion (Figure 7-3). Excessive energy is commonly dissipated with a splash pad or riprap. Some detention practices are designed with the inlet and outlet as the same pipe (Figure 7-4). In this case the water commonly surcharges the sewer pipe and fills the storage pool; water exits when the downstream system hydraulics allow.

Figure 7-3 Inlet Examples

Figure 7-4 Example detention practice with inlet and outlet as the same structure

Forebay

A forebay is a depressed area at the upstream side of a detention practice, designed to remove large sediment by settling or filtering the water. A forebay will also reduce the required frequency of sediment removal in other parts of the practice. Some types of forebays may remove floatable trash and other types of pollutants.

Forebays may be designed with a permanent pool of water or designed to drain the water in between the storm events. Draining of the system may be achieved through a variety of mechanisms, including:

- Infiltration and evapotranspiration
- Construction of an outlet system (i.e. a drain pipe with a restricted flow rate) for the forebay. An outlet system could include a subsurface drain below the forebay
- The wall between the forebay and the primary practice can be constructed of a material that allows water to filter through. For example, an aggregate system such as a gabion basket

Other types of pretreatment systems may be used in place of a forebay, for example a manufactured hydrodynamic separator. The use of a manufactured treatment system is most commonly used for subsurface detention practices.

Site designs that direct all runoff through GSI practices such as bioretention and porous pavement systems have already provided pretreatment before the water reaches the detention practice. In these cases, a forebay may not be needed.

Storage Pool

The storage pool is where the water is temporarily detained. It may be referred to as the practice or primary practice. The storage pool refers the area that is dry or empty between storm events. The storage pool can have multiple stages, for example one sized to hold the smaller storms and a second sized to help store the larger storms.

Construction of the practice often results in significant compaction of the soil. As a result, most detention practices do not infiltrate well. If infiltration is desired, exceptional care and attention is required during design and construction. Practices may also be constructed with an impermeable liner if infiltration is specifically not desired.

The shape of a practice can significantly affect the pollutant-removal efficiency. A length-to-width ratio should be at least 3L:1W for water quality improvements. Figure 7-5 shows practice configurations that may be used to increase the length-to-width ratio and allow for maximum flow path length.

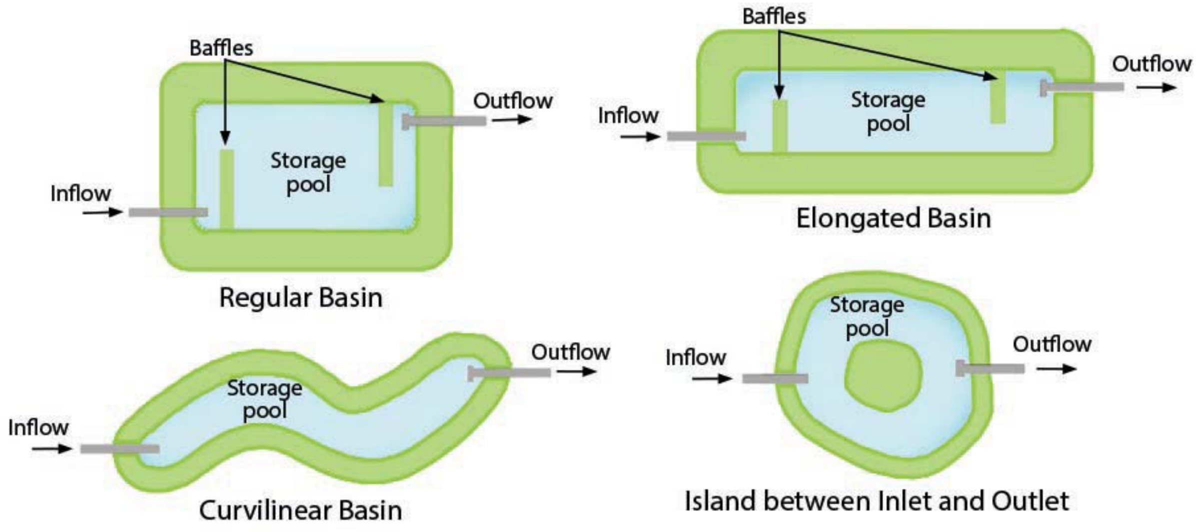

Figure 7-5 Methods of Increasing the Length-to-Width Ratio

Micropool

Extended dry detention practices may incorporate a micropool near the outlet structure. A micropool is a relatively small depressed area that helps prevent resuspension of previously settled sediments and prevents clogging of the low flow orifice.

Permanent Pool

A permanent pool is provided in an extended wet detention practice for improved water quality removals. The theory behind this is that incoming runoff displaces old stormwater from the pond and the new runoff is detained until it is displaced by more runoff from the next storm. Watershed size, soil conditions, and groundwater elevation must be evaluated to ensure the capability of the site to support a permanent wet pond. Consideration should be given for errant vehicles, bicycles, and pedestrians when designing systems with permanent pools.

Figure 7-6 Detention with a Permanent Pool

Outlet

The outlet allow water to exit the storage pool. There are countless outlet configurations that may be used. The most common outlet is composed of a riser pipe with a series of orifices to allow the slow release of water, and an overflow grate on top of the riser pipe for the passage of stormwater from larger rainfall events. There is not a standardized procedure to find the optimal outlet combination. Many different combinations among orifices, culverts, and weirs achieve multiple-event flow control and water quality benefits. Other appurtenances such as trash racks, backflow preventers, odor control systems, and valves may be included to provide a variety of benefits.

Emergency Spillway

An emergency spillway or overflow allows excess water to pass during floods that exceed the practice's design specifications. Emergency overflow systems are commonly a low point in the side of the practice. Care must be taken to direct overflow water to areas that will not cause flooding problems; for example, directing water away from buildings.

Sides and Embankments

Most practices are constructed as depressions in the ground or as buried subsurface detention practices. The sides of the detention are commonly sloped for stability, safety and easy access for maintenance. In some cases, retaining walls may be used.

In some cases, embankments are used to contain the detention practice. An embankment is a raised earthen structure used specially to hold back water, refer to Figure 7-7. Embankments commonly have a clay core, or other impermeable material, and require a specialty design. In some cases, dam safety regulations may be involved if the embankment height exceeds a certain level or the volume of water stored in the practice exceeds a certain volume.

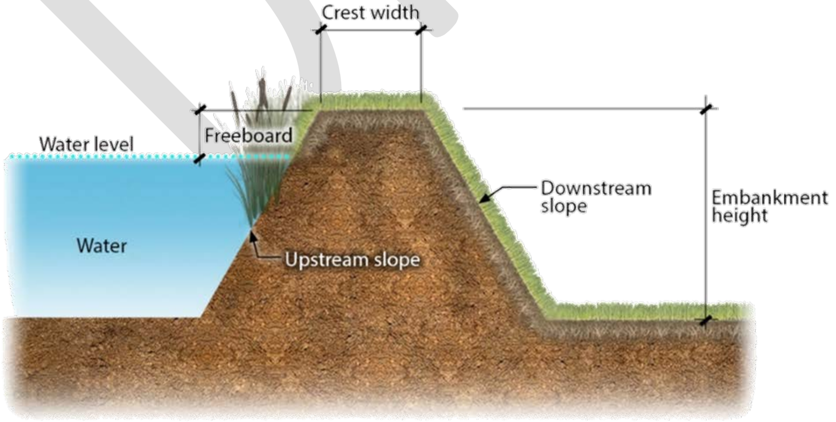

Figure 7-7 Embankment Illustration

Vegetation

Vegetation is commonly used to stabilize the sides and bottom of the practice. Vegetation helps promote infiltration and the plants will consume and transpire water. Plants must be carefully selected based on the frequency and depth of inundation along with the site soil conditions.

Maintenance Access

Maintenance access must be provided for detention practices. Heavy equipment, such as a vacuum truck or excavator are commonly used to clean out the forebay and the outlet structure requires frequent inspection. Designated areas for maintenance equipment must be designed and incorporated into the practice.

7.3 Design Standards

Design standards and requirements are provided in this section. General requirements are presented first, these requirements apply to all detention practices. Following the general requirement section, standards that are unique to each type of detention practice are presented. The design and construction of detention practice must meet all the general requirements and the practices specific requirements.

GSI practices that retain stormwater on-site, for example bioretention and permeable pavement, should be considered first before a detention practice.

7.3.1 Placement on the Site

The following criteria shall be used to define the layout and placement of all detention practices.

- Practices should be located down gradient of disturbed or developed areas on the site. The practice should collect as much site runoff as possible especially from the site’s impervious surface and where other GSI practices are not proposed. Overland flow exceeding the capacity of the site drainage system shall be directed to the practice.
- Practices should not be constructed on steep slopes, nor should slopes be significantly altered or modified to reduce the steepness of the existing slope, for installing a practice.
- Practices should not be constructed within 10 feet of the property line.
- Practices should not be constructed within 10 feet of a municipal sanitary or combined sewer.
- Practices should not be constructed in areas with high quality and/or well-draining soils, which are adequate for installing GSI practices capable of achieving stormwater infiltration and volume reduction.
- Practices shall not be in a stream or any other navigable waters of the United States, including natural (i.e., not constructed) wetlands. Practices shall not be located within a defined 100-year floodplain.

- Practices must have sufficient easements for maintenance purposes. Easements should be sized and located to accommodate access and operation of equipment, spoils, deposition and other activities identified in the development's stormwater management plan.

7.3.2 General Requirements

This section presents general requirements that must be met for all types of detention practices. Requirements are grouped together in categories.

Hydrologic Requirements

Refer to Chapter 2 for site specific design requirements. The hydrologic requirements specified here apply to all detention practices.

- The practice shall be designed to meet the applicable site-specific flow requirements as discussed in Chapter 2.
- The practice shall be designed to safely pass a 100-year storm. If the 100-year storm is not specified as part of the site-specific design requirements an emergency outlet or spillway capable of conveying the 100-year design storm must be included in the design.
- Detention time is defined as the time from when the maximum storage volume is reached to until only 10 percent of the volume remains in the practice.
- The design shall prevent erosion throughout the entire practice including but not limited to the inlet(s), forebay, outlet, emergency overflow, practices sides and embankments. Erosion may be controlled with hard armoring techniques or vegetation.

Layout and Geometry

Requirements

- The lowest elevation within a dry detention practice shall be at least two feet above the seasonal high-water table. If high water table conditions are anticipated, then the design of a wet pond, or constructed wetland should be considered.
- A minimum one (1) foot of freeboard is required above the 100-year flood elevation of the infiltration practice and the low entry elevation of structures near the practice. If building foundation drains are gravity discharged to the infiltration practice, then a minimum of one (1) foot of freeboard is required above the 100-year flood elevation of the infiltration practice and the basement floor elevation of the nearby buildings.
- Detention practices shall be designed with safety considerations including reducing the chance of drowning by the use of warning signs, reducing the maximum depth, or including benching and mild slopes, or any combination thereof, to prevent people from falling in and to facilitate their escape from the practice. To accommodate site constraints, it may be necessary to use retaining

walls, which shall be designed with public safety in mind. With this in mind, side slopes for open detention practices shall comply with the following.

- o Side slopes shall not be steeper than 3H:1V for a minimum of 50 percent of the practice perimeter. Within this portion of the perimeter, terraced slide slopes are allowed however the maximum vertical rise is limited to 18 inches (Wayne County , 2015).

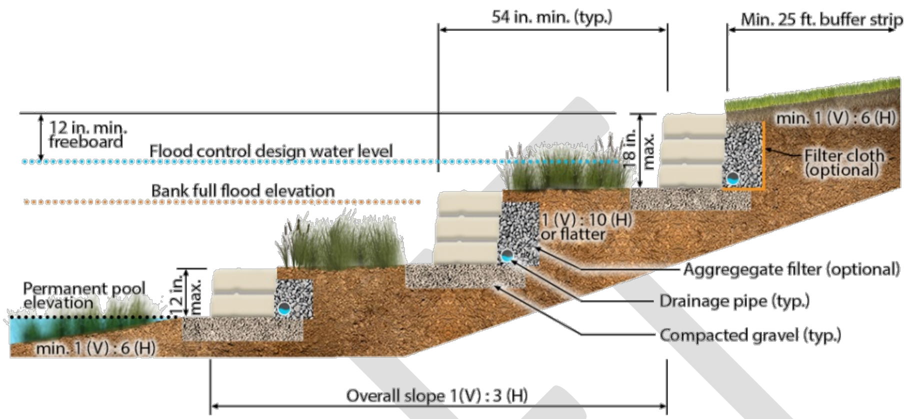

Figure 7-8 Terraced Side Slopes

- o Retaining walls may be provided for up to 50 percent of the practice perimeter. The maximum length of continuous retaining wall is limited to 200 feet unless the width across the practice (measured at the design water level) is less than 50 feet. Safety railing or fences must be provided at the top of all retaining walls per requirements of the Buildings, Safety Engineering and Environmental Department.

- Practices shall be designed to drain toward the outlet.
- If the bottom of the detention practice is designed to infiltrate water or if subsurface drains are provided the minimum slopes and low flow channel are not required.
- Design outlet structures to minimize risk of a person being pushed, pinned or sucked into outlet pipe.
- Avoid situations where the inlet and outlet pipes are directly across from each other and only a short distance apart.
- The detention practice bottom shall be sloped to drain and such slopes shall be sufficient to mitigate against flat spots developing due to construction errors and soil conditions. The minimum transverse slope for the bottom of such practices shall be 2 percent.
- A low flow channel shall be provided from the inlet to the outlet for practices intended to be dry between storm events.
  - o The low flow channel shall have a minimum depth of 1 foot, side slopes no steeper than 3H:1V, a minimum width of 6 feet, and sloped sufficient to mitigate against flat spots developing due to construction errors and soil conditions.

    - The low flow channel may be paved with concrete. When constructed of concrete the low flow channel shall be 6-inch minimum thickness and reinforced to accommodate temperature stresses.
- Permanent access must be provided to the forebay and outlet. It shall be at least nine feet wide, have a maximum slope of 15 percent, and be stabilized for vehicles.

Preferred Design Elements

- Additional considerations for side slopes includes:
    - 6H:1V is the preferred maximum side slope (ASCE, 2014),
    - 4H:1V is the maximum recommended side slope for grassed areas requiring regular mowing, and
    - 10H:1V is the recommended side slope where space is available.
    - 20H:1V is the minimum recommended side slope.
- Irregularly shaped practices are acceptable and encouraged to improve site aesthetics and environmental benefits
- Trash/safety racks should be considered on a case-by-case basis. Hinged racks facilitate cleaning.

Pretreatment

A forebay or other pretreatment system is highly recommended at all major inflow points to capture coarse sediment, prevent excessive sediment accumulation in the main practice, and minimize erosion by inflow. Stormwater runoff that has already passed through another GSI practice does not need to pass through a second pretreatment device.

Design elements for forebay include:

- Size the volume of the forebay to contain 10 percent of the water quality treatment volume.
- Forebays shall have a minimum length of 10 feet.
- Forebays shall have a depth of 4 to 6 feet.
- The bottom of the forebay should be hardened (for example with concrete or grouted riprap) to make sediment removal easier.
- Physically separate the forebay from the primary storage pool with a berm, gabion wall, or other divider.
- Flows exiting the forebay must be non-erosive.
- Install a permanent vertical marker that indicates the sediment accumulation depth.

The forebay storage volume counts toward the overall storage volume required if the forebay is dewatered between rain events.

Outlet

Outlets for detention can be designed in a wide variety of configurations. Most outlets use modified boxes or riser pipes made of concrete or corrugated metal. These structures can be designed to control different storms using several orifices or pipes; for example, a small inlet to control the water quality volume, an orifice to control a 2-yr storm, and a larger orifice to control a 10-yr storm. This larger flow is usually controlled by stormwater flowing in through the top of the structure. If risers are used, an antivortex design may be necessary for flow entering the top of the pipe. Larger flows are usually handled by an emergency spillway. Because of flow restriction requirements, low flow outlets are often very small. The design must guard against clogging small outlets.

General Requirements

- The detention practice shall be designed with an outlet control system sized to meet the hydrologic requirements.
- Outlet shall be designed to retain floatables, such as debris, oil and grease within the practice. Acceptable floatables control devices include perforated pipes, skimmers, baffles, inverted pipes and other devices approved by the Department.
- Inlet and outlet pipes and risers shall be constructed of reinforced concrete, corrugated metal or smooth lined corrugated plastic pipe. The minimum diameter for riser pipes less than 4-ft. in height is 24-inch. Riser pipes greater than 4-ft. in height shall be a minimum 48-inch diameter.
- Riser pipes constructed of corrugate metal or smooth lined corrugated plastic must be set into a cast-in-place concrete base. Riser pipes constructed of reinforced concrete may be set into a cast-in-place concrete base or properly grouted to a pre-cast concrete base.
- Outlets must be placed near or within the side of the practice to provide ready maintenance access.

Orifice Requirements

- Orifices shall be 0.75-inch minimum diameter.
- The outlet shall be designed to resist plugging. Orifices smaller than 4 inches in diameter shall be protected from clogging with an aggregate jacket around the riser pipe, subsurface drains, specialty screens or other approved devices.
    - Aggregate Jacket Around Perforated Riser Pipe. The open graded aggregate shall be placed around the around the riser pipe. The orifice configuration shall be wrapped with a hard wire mesh with an appropriate opening size to prevent any stones from passing through the orifice. The recommended open graded aggregate is a 3-inch diameter washed stone placed immediately adjacent to the riser pipe with an outer blanket of MDOT 6A stone. The side slope of the stone blanket is typically 2H:1V.
    - Subsurface Drainage Dewatering. Where subsurface drains are placed under the bottom of the detention practice and used for dewatering the orifice limiting the flow rate shall be placed inside a manhole structure. No additional measures are required to prevent clogging since flow entering the subsurface drainage system is presumed to have been filtered through a soil or other media layer.
    - Screens. Specialty screens designed to resist clogging may be used.
- When the orifices are in a corrugated metal material the holes shall be pre-drilled prior to galvanizing.

Figure 7-9 Aggregate Jacket Example

Specialty screen protecting orifice plate from clogging, outside (left); orifice plate, inside (center); clogged poorly designed screen (right)

Figure 7-10 Screens Example

Trash Rack Requirements

Trash rack serve two purposes; (1) to prevent conveying trash and debris downstream, and (2) to act as a safety grate for people and large animals.

- The top of risers and overflow structures shall be equipped with a trash rack.
- Openings shall be a maximum of 4 inches. Recommended opening are 1.5 inches.
- Trash racks are recommended to be installed at a slight angle (approximately 15 degrees) to prevent ice formation and to minimize clogging.

Consider maintenance of the structure and potential access by the public when selecting the type of trash rack. For example, a close mesh grate will be more appropriate in high pedestrian traffic but will require more frequent maintenance as it will catch smaller debris. Trash racks of sufficient size should always be provided on an outlet structure so that they do not interfere with the hydraulic capacity of the outlet.

Piping Downstream of the Outlet Structure

- The minimum pipe size downstream of the flow controls in the practice outlet shall be 12-inch.
- An anti-seepage collar shall be provided on each outlet pipe and watertight joints shall be used on the pipe segment near the anti-seepage collar.
- When connecting to a combined sewer system a backflow preventer and odor trap shall be provided downstream of the outlet structure.

Emergency Spillway

- The emergency spillway elevation shall be set at the elevation of the maximum detention practice design volume.
- The emergency spillway shall be designed to pass the maximum design flow tributary to the detention practice.
- The emergency overflow must be armored to prevent erosion.
- When an embankment is used around the detention practice, the embankment shall be designed to prevent water seepage.

Pumped Outlets

Gravity outlets are preferred over pumped outlets. If a detention practice is designed to include a pumped outlet the following requirements apply

- A minimum of two pumps should be provided in any pumped outlet system. The pumps shall be designed such that the maximum pumping capacity does not exceed the allowable release rate. A backup pump shall be provided.

Vegetation

- Plant vegetation is required for all types of wet detention practices to control erosion and enhance sediment entrapment.
- A landscaping plan is required for open detention practices, due to the importance of the vegetation to the function of the entire system. At a minimum, the landscape plan shall include the following:
    - Existing site conditions and vegetation (e.g. trees 6-inch caliper and larger) that may be affected by the project;
    - Plan view of the open detention practice, including one-foot grading contours;
    - Elevations in the open detention practice, including the detention practice bottom elevation and all the maximum water surface elevations based on the hydrologic requirements;
    - Identification of planting zones based on levels of inundation; and
    - Vegetation selection, plant spacing and applicable depths.
- Woody vegetation may not be planted on nor allowed to grow within 15 feet of the toe of an embankment.
- Woody vegetation may not be planted on nor allowed to grow within 25 feet of the emergency overflow.

7.3.3 Extended Dry Detention

All the general requirements (Section 7.3.2) apply along with the additional requirements discussed in this section.

- The water quality treatment volume shall be dewatered in a minimum of 24 hours.
- The dewatering duration of the practice at full stage shall occur between 40 and 72 hours.
- The practice shall gradually expand from the inlet, toward the outlet.
- The length to width ratio shall be a minimum of 3:1.
- Distances of flow paths from inflow points to outlets should be maximized.
- If site conditions inhibit construction of a long, narrow practice, baffles consisting of earthen berms or other materials can be incorporated into the design to lengthen the stormwater flow path.
- A two-stage design is recommended with a 1.5 to 3.0 ft. deep bottom stage and a 2 to 6 ft. deep upper stage.
- A wetland marsh (micropool) created in the bottom stage will help remove soluble pollutants that cannot be removed by settling.

7.3.4 Extended Wet Detention

All the general requirements (Section 7.3.2) and extended dry detention requirements (Section 7.3.3) apply along with the additional requirements discussed in this section.

Requirements

- For quality, the permanent pool shall be at least the water quality treatment volume for the drainage area.
- A minimum 3 ft. deep permanent pool shall be provided (less could allow insect breeding and wind re-suspension of settled particles). The maximum permanent pool depth is 10 ft. (could lead to thermal stratification in the pond and anaerobic conditions in the deep water) (WEF and ASCE/EWRI, 2012). The volume of the permanent pool does not satisfy any portion of the required flood control storage volume.
- A mean depth of 3 to 10 feet shall be provided. A mean depth of the permanent pool is calculated by dividing the storage volume by the surface. (WEF and ASCE/EWRI, 2012).
- A minimum depth of the open water area shall be 6 feet to prevent emergent plant growth in this area (WEF and ASCE/EWRI, 2012).
- A maximum depth of the open water area shall be 13 feet to reduce the risk of thermal stratification (WEF and ASCE/EWRI, 2012).
- A littoral zone shall be established around the perimeter of the permanent pool to promote the growth of emergent vegetation along the shoreline and deter individuals from wading (WEF and ASCE/EWRI, 2012).
- The bench for the littoral shall be at least 10 feet wide with a water depth of 0.5 to 1.0 ft. The total area of the aquatic bench shall be 25 to 50% of the permanent pool's water surface area. (WEF and ASCE/EWRI, 2012). An aquatic bench is not required in forebays.
- If clay, synthetic or plastic liners are used to minimize seepage through the bottom of the permanent pool the liner shall be covered with gravel or other material to provide footing and/or utilize other measures to enable egress.
- A permanent buffer strip of vegetation extending at least 15 feet in width beyond the freeboard elevation must be provided around the practice. The slope of the buffer strip should be 6H:1V or flatter.
- The permanent pools shall have a drain pipe equipped with an adjustable valve that can completely drain the permanent pool within 24 hours. Valve controls shall be located at a point where it will not normally be inundated and can be operated in a safe manner.

Design Considerations

- The area necessary for a permanent pool is generally one to three percent of its drainage area.
- The presence of a mechanical aerator, such as a fountain in the middle of the pond, may be used to make the site more attractive, deter the growth of unwanted vegetation, and make the habitat more suitable for fish. If aerating devices are used as part of a stormwater management system, they should be designed to minimize disturbance of bottom sediments.
- Extended wet detention practices require groundwater or a dry-weather base flow if the permanent pool elevation is to be maintained year-round.

- The designer should consider the overall water budget to ensure that the permanent pool will not be lost due to evaporation, evapotranspiration, and seepage (unless the pond is lined). High exfiltration rates can initially make it difficult to maintain a permanent pool in a new practice, but the bottom can eventually seal with fine sediment and become relatively impermeable over time. However, it is best to seal the bottom and the sides of a permanent pool if the pool is located on permeable soils and to leave the areas above the permanent pool unsealed to promote infiltration of the stormwater detained.

7.3.5 Subsurface Detention

Subsurface detention systems consist of one or more underground pipes or structures designed to provide the required storage volumes. All the general requirements (Section 7.3.2) apply along with the additional requirements discussed in this section.

- A pretreatment system is required with all subsurface detention practices.
- All subsurface detention practices shall have a means to inspect and maintain the entire system.
- Subsurface detention practices shall be designed and constructed for a minimum HS-20 loading.
- All manufacturer recommendations shall be adhered to for subsurface detention practices.

Figure 7-11 Subsurface Detention During Construction

7.3.6 Parking Lot Detention

Parking lot storage is a stormwater quantity control method allowing shallow ponding within paved portions of the parking lot. The following criteria shall apply:

- Ponding in parking or traffic areas shall be designed for a maximum ponding depth of 12 in. for all storms up to and including the 100-year event. Flood routing or overflow to a designated conveyance system must occur after the maximum depth is reached.
- A site with a parking lot storage practice shall employ a separate water quality treatment practice. This practice may be located either downstream of the parking lot or integrated into the medians, landscaping, or other pervious areas of the parking lot.
- Public education signs shall be posted clearly indicating that the parking lot is intended to flood during rain events.

Is parking lot detention an option that DWSD wants to offer? This would be a cheaper option for many parking lots (compared to subsurface detention). This is an option that other communities (e.g. City of Columbus OH) allow. The photo below is from Denver CO (depth is about 24 to 30 inches in the photo). Suggest this is discussed with Legal.

Figure 7-12 Parking Lot Detention Example

7.3.7 Practices Used for Harvest and Reuse

Urban water harvesting and reuse may be developed for urban areas and are mainly suitable for non-drinking purposes such as irrigation, industrial uses, and water features. This section does not cover potential uses for water reuse in growing crops or in aquaculture. Refer also to the Rainwater Harvesting chapter (Chapter 11) for additional information. Detention practices intended for water harvesting and reuse should be designed to meet the requirements of extended wet detention practices (Section 7.3.4). Additional design considerations apply and are summarized below.

Requirements

- Provide a brief narrative discussing the operation of the water reuse.
- Estimate the water storage requirements.
- Provide water budget calculations showing the average monthly water reuse and losses due to seepage and evapotranspiration. The computations shall be based on a minimum of 10 years of representative daily rainfall and climatic data. Winter time conditions and plans shall be documented.
- The volume of the permanent pool and drawdown areas shall be ignored for the purposes of sizing the storage pool
- A permanent pool depth shall be maintained below which no pumping occurs to prevent resuspension of sediment.
- All operations must conform to the Michigan Plumbing Code.
- The practice may be subject to additional requirements by the Department and BSEED.

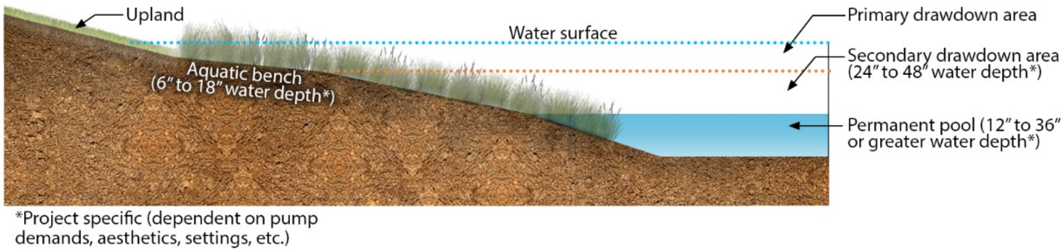

Figure 7-13 Hydraulic Profile of Detention Practice for Reuse

Design Considerations

- Reuse of stormwater from a pond treating runoff from potential stormwater hotspots may pose a public safety and welfare concern, as well as may be cost prohibitive to pre-treat if special filter devices are required.
- Reuse systems may require a supplemental water-supply system for irrigation needs when runoff is not available.
- Multiple aquatic benches may be necessary for ponds that experience repeated drawdown due to irrigation reuse. An alternative to multiple aquatic benches

would be mild side slopes of 5H:1V from the bench downward to the permanent pool elevation, then grade downward as necessary. These considerations are dependent on aesthetics, adjacent land use (residential vs. commercial, etc.), and objectives for operations and maintenance.

- Ponds that experience repeated bounce or drawdown due to irrigation reuse may create an environment for invasive vegetation species.

7.4 Calculations and Sizing

This section presents a brief review of hydrology and hydraulics for detention system sizing. Spreadsheets are easily developed for the sizing calculations. A variety of computer software programs are also available.

7.4.1 Detention Volume Sizing

Design of a detention practice involves routing the inflow hydrograph through the practice. The following data will be needed to complete storage design and routing calculations:

1) inflow hydrograph for the necessary design storms (refer to Chapter 2 for regulatory requirements);
2) stage-storage curve for proposed storage practice; and
3) stage-discharge curve for all outlet control structures.

Using this data, a design procedure is used to route the inflow hydrograph through the storage practice to establish an outflow hydrograph. If the desired outflow results are not achieved, practice and outlet geometry are varied to yield new stage-storage and stage-discharge curves and the routing procedure is redone until the desired outflow hydrograph is achieved.

Flood Routing

The fundamental equation for routing water through a proposed practice is based on the principle of mass conservation. The change of volume of water in storage in the practice is described by the equation:

$$I - Q = \frac{dS}{dt} \tag{7.1}$$

where I = inflow rate
Q = outflow rate
S = storage volume
t = time

For a finite time period, Δt, Equation (7.1) can be written in finite difference form and rearranged as:

$$(I_1 - I_2) + \left(\frac{2S_1}{\Delta t} - Q_1\right) = \left(\frac{2S_2}{\Delta t} + Q_2\right) \tag{7.2}$$

where I_1 = inflow rate at start of the time period
I_2 = inflow rate at the end of the time period
Δt = duration of the time period
S_1 = storage at the beginning of the time period
S_2 = storage at the end of the time period
Q_1 = outflow rate at the beginning of the time period
Q_2 = outflow rate at the end of the time period

Solving Equation (7.2) involves an approach often referred to as the Modified-Puls method. The Modified-Puls method provides a numerical technique for solving this differential equation. Two fundamental assumptions are included in this method: (1) that the storage depends only on the outflow rate, and (2) that the water surface in the practice is horizontal (known as Level Pool Routing).

Before solving Equation (7.2) a *stage-storage* relationship and a *stage-discharge* relationship need to be developed.

> *Stage-Storage*. This is the relationship between the storage capacity of the system relative to the depth of water. This relationship is simply a function of the physical geometry of the practice. Additional details on the relationship are provided on Page 7-25.
>
> *Stage-Discharge*. This relationship describes the rate of discharge from the practice relative to the depth of water. Additional details on the relationship are provided on Page 7-25.

The stage-storage and stage-discharge relationships are then combined, or used together, to relate the storage to the discharge of the practice. Refer to Table 7-1 as an example. Interpolation between data points may be done with either a semi-graphical approach or using a spreadsheet.

Example Detention Basin
Assume 3-inch diameter orifice at bottom of practice.

Stage (ft)	Surface Area (sf)	Storage (cf)
0	2,398	0
1	3,135	2,760
2	3,951	6,296
3	4,843	10,686
4	5,814	16,008
5	6,861	22,338

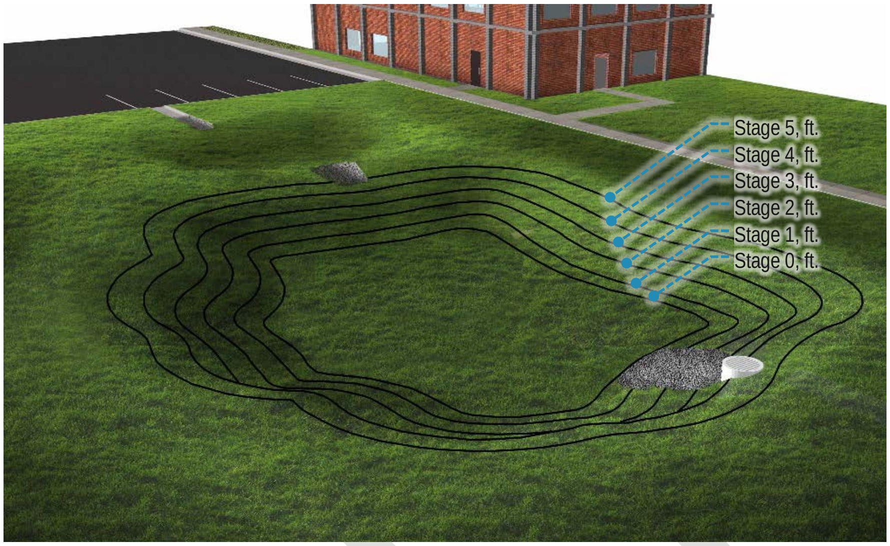

Figure 7-14 Example Detention Basin

Table 7-1 Storage-Discharge Relationship Example

(1) Stage h (ft)	(2) Discharge Q (cfs)	(3) Storage S (ft^3)	(4) [(2S/Δt)+Q] (cfs)
0.0	0.00	0	0.0
0.5	0.17	1,288	4.5
1.0	0.24	2,760	9.4
1.5	0.29	4,426	15.0
2.0	0.33	6,296	21.3
2.5	0.37	8,379	28.3
3.0	0.41	10,686	36.0
3.5	0.44	13,226	44.5
4.0	0.47	16,008	53.8
4.5	0.50	19,042	64.0
5.0	0.53	22,338	75.0

Given the inflow hydrograph, the stage-storage, and the stage-discharge information the process for solving Equation (7.2) is then as follows:

1. From the given stage-storage and stage-discharge relationships obtain a storage-discharge (S versus Q) relationship. Refer to Table 7-1 columns 1 thru 3.
2. Select a time increment, Δt. Calculate the quantity [(2S/Δt) + Q], refer to column 4 in Table 7-1.
3. Record the selected time increment, Δt. In Table 7-2 columns 1 thru 3 are filled in.

4. Record the inflow hydrograph information and for each time step computation calculate ($I_1 + I_2$), (Table 7-2 column 4 thru 6)
5. Calculate [$(S_2/\Delta t) + Q_2$] from Equation (7.9), Table 7-2 column 7.
6. Obtain Q_2 from the storage-discharge relationship (Table 7-1). In the example, the result is recorded in column 9 of Table 7-2.
7. To proceed to the next time step, first calculate [$(2S_2/\Delta t) - Q_2$]. This is calculated by taken [$(2S_2/\Delta t) + Q_2$] - $2Q_2$. In other words, looking at Table 7-2 Column 10 equals Column 8 minus 2 times Column 9. The value of [$(2S_2/\Delta t) - Q_2$] calculated at any time step will become [$(2S_1/\Delta t) - Q_1$] for the next time step.
8. Repeat the same procedure until the routing is completed.

Table 7-2 Example Detention Practice Routing

(1) Time step	(2) t_1 (min)	(3) t_2 (min)	(4) I_1 (cfs)	(5) I_2 (cfs)	(6) I_1+I_2 (cfs)	(7) $[(2S_1/\Delta t)-Q_1]$ (cfs)	(8) $[(2S_2/\Delta t)+Q_2]$ (cfs)	(9) Q_2 (cfs)	(10) $[(2S_2/\Delta t)-Q_2]$ (cfs)
1	0	10	0.0	0.5	0.5	0.0	0.5	0.02	0.5
2	10	20	0.5	2.5	3.0	0.5	3.5	0.13	3.2
3	20	30	2.5	3.8	6.3	3.2	9.5	0.24	9.0
4	30	40	3.8	4.1	7.9	9.0	16.9	0.30	16.3
5	40	50	4.1	3.8	7.9	16.3	24.2	0.35	23.5
6	50	60	3.8	3.2	7.0	23.5	30.5	0.38	29.8
7	60	70	3.2	2.6	5.8	29.8	35.6	0.41	34.7
8	70	80	2.6	2.1	4.7	34.7	39.4	0.42	38.6
9	80	90	2.1	1.7	3.8	38.6	42.4	0.43	41.5
10	90	100	1.7	1.3	3.0	41.5	44.5	0.44	43.6
11	100	110	1.3	0.9	2.2	43.6	45.8	0.45	44.9
12	110	120	0.9	0.6	1.5	44.9	46.4	0.45	45.6
13	120	130	0.6	0.3	0.9	45.6	46.5	0.45	45.6
14	130	140	0.3	0.2	0.5	45.6	46.0	0.45	45.1
15	140	150	0.2	0.1	0.2	45.1	45.3	0.44	44.4
16	150	160	0.1	0.0	0.1	44.4	44.5	0.44	43.6
17	160	170	0.0	0.0	0.0	43.6	43.6	0.44	42.7
18	170	180	0.0	0.0	0.0	42.7	42.7	0.44	41.8
19	180	190	0.0	0.0	0.0	41.8	41.8	0.43	41.0
20	190	200	0.0	0.0	0.0	41.0	41.0	0.43	40.1
21	200	210	0.0	0.0	0.0	40.1	40.1	0.43	39.3

Stage Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a practice. For regular-shaped practices the stage-storage relationship can be obtained from the geometry of the practice. For example, for a trapezoidal detention practice that has a rectangular base the relationship between the volume (or storage) and the flow depth *is:*

$$S = LWd + (L + W)zd^2 + \frac{4}{3}z^2d^3 \qquad (7.3)$$

where S = storage volume
L = rectangular base length
W = rectangular base width
z = side slope
d = depth

Storage practices are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. The data for this type of curve is usually developed using a topographic map. For irregular shaped detention practices, the volume between two contours may be estimated by taken the average of the surface areas times the height.

$$S = (h_2 - h_1)\frac{A_{s1} + A_{s2}}{2} \qquad (7.4)$$

where A_{s1} = surface area at h_1
A_{s2} = surface area at h_2
h = elevation
S = storage volume

A more accurate relationship is given by:

$$S = \frac{(h_2 - h_1)}{3}\left(A_{s1} + A_{s2} + \sqrt{A_{s1}A_{s2}}\right) \qquad (7.5)$$

Stage Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage practice. A typical storage practice has two or more outlets. The principal outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency outlet or spillway.

A typical outlet structure may include culverts, weirs, orifices, or a combination of all three. The emergency outlet or spillway should be sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. When the outlet and spillway are designed, consider the potential threat to downstream life and property.

The stage-discharge curve should reflect the discharge characteristics of both the principal and emergency outlets. Develop a composite stage-discharge curve, which combines the discharge rating curve for all components of the outlet control structure.

7.4.2 Outlet

A typical outlet structure may include a riser structure, culverts, weirs, orifices, or a combination of all three. There is not a standardized procedure to find the optimal outlet combination for a detention practice. Many different combinations of orifices, culverts and weirs achieve multiple-event outlet structures.

Risers

A *riser* is a vertical pipe with perforations (i.e. orifices) strategically placed and sized for discharging water under specific conditions. Risers may also be referred to as a *stand pipe* if it has a circular cross section and an *inlet box* if it has a rectangular cross section. Often the top of the riser is used as an overflow system in which case they are typically at least 24-inches wide. Separate smaller diameter riser pipes may also be constructed next to a larger overflow structure.

Risers operate hydraulically as weirs when the head over the structure is low. When fully submerged, risers behave as an orifice. Design calculations should reflect these varying conditions.

Figure 7-15 Typical Riser Pipe with Stone Jacket

Orifice Hydraulics

An orifice is an opening or hole. Orifices may be placed vertically, e.g. on the side of a riser pipe, or horizontally. Both vertical and horizontal orifices behave like a weir when water is so shallow that the opening is not entirely submerged.

When the water depth above a horizontal orifice is less than half the diameter of the orifice, the orifice behaves like a weir (in this case the weir length is equal to the circumference of the orifice). The actual transition from weir flow to orifice flow is not well established. The *half the diameter* is a simple rule-of-thumb.

When fully submerged, the flow rate through an orifice is described by the orifice equation as:

$$Q = CA\sqrt{2gh} \tag{7.6}$$

where Q = flow rate, cfs
C = orifice discharge coefficient, dimensionless
A = area of the orifice opening, sq. ft.
g = gravitational acceleration, 32.2 ft/s^2
h = depth of water above the centerline of the orifice, ft

Orifice coefficients are provided in Table 7-3 (Brater & King, 1976).

Table 7-3 Orifice Coefficients

Shape	Orifice Coefficient
circular	0.614
square with vertical walls	0.616
rectangle, side ratio of 4:1, long side in vertical direction	0.626
rectangle, side ratio of 4:1, long side in horizontal direction	0.627
rectangle, side ratio of 10:1, long side in vertical direction	0.637
rectangle, side ratio of 10:1, long side in horizontal direction	0.637
triangle	0.615

Weir Hydraulics

Weirs are openings in barriers to direct or control the flow of water. Weirs operate under open channel conditions, i.e. they are not pressurized. The top surface of water flowing over a weir is open.

Weirs fall into two broad categories based on the length of the crest (i.e. the length of the weir in the direction of the flow); broad crested and sharp crested. The length of a broad crested weir extends far enough in the flow direction to cause the occurrence of critical flow depth over the crest. Sharp crested weirs have a free-falling nappe and the critical flow depth occurs off the crest. Most weirs on outlet control structures fall into the category of a sharp crested weir.

Figure 7-16 Weir Examples

Flow over a weir is governed by the weir shape. The discharge rate of a rectangular weir is described by Equation (7.7) and a triangular shaped weir is given by Equation (7.8). Discharge through a trapezoidal shaped weir is calculated as the sum of the rectangular section and the triangular section.

$$Q = CLh^{3/2} \quad \textit{rectangular weir shape} \tag{7.7}$$

$$Q = Ch^{5/2}\tan\left(\frac{\theta}{2}\right) \quad \textit{triangular weir shape} \tag{7.8}$$

where Q = discharge, cfs
C = weir coefficient, dimensionless
L = weir length (rectangular weir), ft
h = depth of water above the weir crest, ft
θ = center angle of triangular weir

The weir discharge coefficient (C) for a broad- or sharp-crested weir ranges between 2.65 and 3.10. A value of C=3.0 is recommended for sharp crested weirs and C=2.65 is recommended for broad-crested weirs unless specific weir configurations and downstream tailwater conditions are known. The Handbook of Hydraulics by E. F. Brater and H. W. King is a good reference for weir coefficients (Brater & King, 1976).

Reverse Sloped Pipes

Reverse sloped pipes may be used with extended wet detention practices where one end of the pipe extends to the bottom of the permanent pool and the other end of the pipe discharges into the control structure. In this way, water is removed from the bottom of the permanent pool. Flow through a reverse sloped pipe may be described by a variation to the energy equation:

$$Q = \frac{A\sqrt{2gh}}{\sqrt{1 + K_e + K_b + K_c L}} \tag{7.9}$$

where Q = discharge, ft^3/s
A = cross-sectional area of the pipe, ft^2
g = gravitational acceleration, 32.2 ft/s^2
h = depth (head) above discharge end of pipe, ft
K_e = entrance loss coefficient, dimensionless
K_b = bend loss coefficient (zero for no bends), dimensionless
K_c = head loss coefficient for pipe, dimensionless
L = pipe length, ft

Emergency Overflow

Emergency overflows are commonly constructed as low points in the sides surrounding the permanent pool. The overflows are typically designed wide to keep the head relatively low. For calculation and sizing the emergency overflow is treated as a weir. A broad crested weir is commonly assumed when the overflow is constructed through an earthen low point. A sharp crested weir may be appropriate when the overflow is controlled by a riser. When designing as an earthen low point, attention should be paid to the water velocity and an appropriate armoring technique applied to guard against erosion.

7.5 Operation and Maintenance

As discussed in the Regulatory Requirements (Chapter 2), an operation and maintenance plan is required for all stormwater practices. A crucial step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage practices. To assure acceptable performance and function, storage practices that require extensive maintenance are discouraged. Practices should be designed to minimize the following maintenance problems that are typical with urban detention practices:

An operation and maintenance plan must be prepared and submitted for review and approval. Refer to Chapter 2, Regulatory Requirements, for additional details.

- weed growth,
- grass and vegetation maintenance,
- sedimentation control,
- bank deterioration,
- standing water or soggy surfaces,
- mosquito control,
- blockage of outlet structures,
- litter accumulation, and
- maintenance of fences and perimeter plantings.

Proper design should focus on easing the maintenance burden by addressing the potential for problems to develop; for example:

- Address weed growth and grass maintenance by constructing side slopes that can be maintained using available power-driven equipment (e.g., tractor mowers).
- Control sedimentation by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.
- Control bank deterioration with protective lining or by limiting bank slopes.
- Eliminate standing water or soggy surfaces by sloping practice bottoms toward the outlet, constructing low-flow pilot channels across practice bottoms from the inlet to the outlet, or constructing an underdrain. If the detention practice is constructed to dewater within 72-hours, mosquitoes should not be a problem.
- Select outlet structures to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and should be avoided). Ice accumulation should also be considered.
- Locate the practice for easy access so that maintenance can be conducted on a regular basis to address litter and damage to fences and perimeter plantings.

Inspection and maintenance are key to ensure the proper function and aesthetics of detention and retention practices. The table below lists specific operation and maintenance tasks.

Task	Frequency	Indicator that maintenance is needed	Maintenance notes
Forebay inspection	2-4 times/year	Internal erosion or excessive sediment, trash, or debris accumulation	Check for sediment accumulation to ensure that forebay capacity is as designed. Remove any accumulated sediment.
Practice inspection	1 time/year	Excessive sediment, trash, and/or debris accumulation in the practice	Remove any accumulated sediment. Adjacent pervious areas might need to be regraded.
Outlet inspection and maintenance	2-4 times/year	Accumulation of litter and debris in practice, large debris around outlet, internal erosion	Remove litter, leaves, and debris to reduce the risk of outlet clogging and to improve practice aesthetics. Erosion should be repaired and stabilized.
Mowing	2-12 times/year	Overgrown vegetation on embankment or adjacent areas	Frequency depends on location and desired aesthetic appeal.
Embankment inspection	1 time/year	Erosion at embankment	Repair eroded areas and revegetate.
Remove and replace dead vegetation	2-4 times/year	Dead plants or excessive open areas in practice	Within the first year, 10% of plants can die. Survival rates increase with time.
Temporary watering	1 time/2-3 days for the first 1-2 months	Until establishment and in severe drought	Watering after the initial year might be required.
Nuisance wildlife management	Biweekly or as needed	Animals, feces, or burrows evident in or around practice. Excessive mosquitos.	Maintain diverse vegetated shelf around entire practice. Eliminate monocultures and replace with diverse, flowing vegetation. Employ qualified wildlife management professionals if needed.
Fertilization	1 time initially	Upon planting	One-time spot fertilization for first year vegetation.

7.6 Design Checklist

7.7 References

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8. Bioretention

Bioretention is a term used to describe both a specific practice and to include a broader set of GSI practices that incorporate a common set of design elements. All are included in this category because they use vegetation and soil as an integral part of the practice design. Examples of different configurations of bioretention practices include: small rain gardens at residential sites; larger discrete practices at commercial sites; linear systems or bioswales placed between the sidewalk and the road; curb extensions or bump-outs into the roadways; raised planter boxes typically accepting water from roof drains; and flushed planter boxes typically located in pedestrian areas.

Contents

8.1 Introduction

Bioretention practices are landscaped, shallow depressions that capture and temporarily store stormwater runoff. Runoff intercepted by the practice is then filtered through the soil (often engineered soil filter media). Pollutants are removed through a variety of physical, biological, and chemical treatment processes. Bioretention practices are the most commonly implemented GSI technique because they help to mimic predevelopment hydrologic conditions, enhance biodiversity and water quality, and can be easily incorporated into both new and existing developments (Davis, Hunt, Traver, & Clar, 2009). Bioretention practices usually consist of a pretreatment system, surface ponding area, mulch layer, soil filter media, aggregate storage layer, vegetation, and subsurface drainage system.

8.1.1 Major Components

Advantages

- Flexible layout and easy to incorporate in landscaped areas
- Very effective at removing pollutants and reducing runoff volumes
- Generally, one of the more cost-effective stormwater management options
- Relatively modest maintenance activity costs
- Can contribute to better air quality and help reduce urban heat island impacts
- Can improve property values and site aesthetics through attractive landscaping

Limitations

- Maintenance requirements can be higher initially until vegetation is established
- Requires careful selection and establishment of plants
- Limited impervious drainage area

The major components of a bioretention practice shown in Figure 8-1 and described below include:

Surface Ponding Layer – surface ponding may be included to increase flow retention and augment peak discharge control.

Vegetation – along with the soil filter media, vegetation is a critical component of a bioretention practice. Vegetation takes up nutrients and water, and enhances and maintains the infiltration capacity of the soil. The practice is planted with small- to medium-sized vegetation including ground covers, grasses and forbs that can withstand urban environments and tolerate both periodic inundation and extended dry periods. Shrubs and trees may also be included in bioretention practices. Plantings provide habitat for beneficial pollinators and aesthetic benefits for stakeholders and can be customized to attract butterflies or specific bird species.

Soil Filter Media – typically a sand and loam mix that supports vegetation growth and removes pollutants from stormwater runoff. The depth of the soil filter media can vary greatly, but is commonly 12 to 36 inches deep. The surface of the soil filter media is generally flat except in the case of bioswales.

Sides – the sides of the bioretention practice are extended up to integrate the practice into the surrounding topography. Sides are designed to contain the stormwater runoff, be aesthetically attractive, provide safety to the public, and provide easy maintenance access.

Aggregate Storage Layer – a layer of open-graded aggregate may be added below the soil filter media to provide additional stormwater storage volume. The water storage potential in an aggregate layer is approximately twice that of soil filter media layer of the same thickness. The presence of a layer of aggregate may impede deep vegetation root growth.

Subsurface Drainage – bioretention practices need to have adequate drainage to allow for infiltration into the subgrade while still supporting the vegetation community and preventing nuisance pests such as mosquitos. Underdrains are incorporated as needed to support these uses.

Liner – a special layer placed on the sides, the bottom, or both. Liners may be impermeable or permeable and are incorporated based on the type of practice being designed and site conditions.

Inlet and Pretreatment – a hydraulic structure designed to direct the stormwater runoff into the bioretention practice. Pretreatment serves to remove coarse sediment and oftentimes floatable trash. Inlets and pretreatment systems also incorporate energy dissipation to protect against scour and erosion. Inlets and pretreatment are often incorporated together into a single structure, but may be separate elements if necessary.

Outlets and Overflows – hydraulic structures designed to let water out of the bioretention practice. Outlets may be provided for excess surface water and for subsurface drainage. Overflows may be provided to prevent water from overtopping the practice and flooding surrounding areas.

Maintenance Access – a dedicated access to the practice to allow for efficient and cost-effective maintenance. Maintenance access is needed for the surface features and to the subsurface drainage system, inlet, pretreatment and outlet.

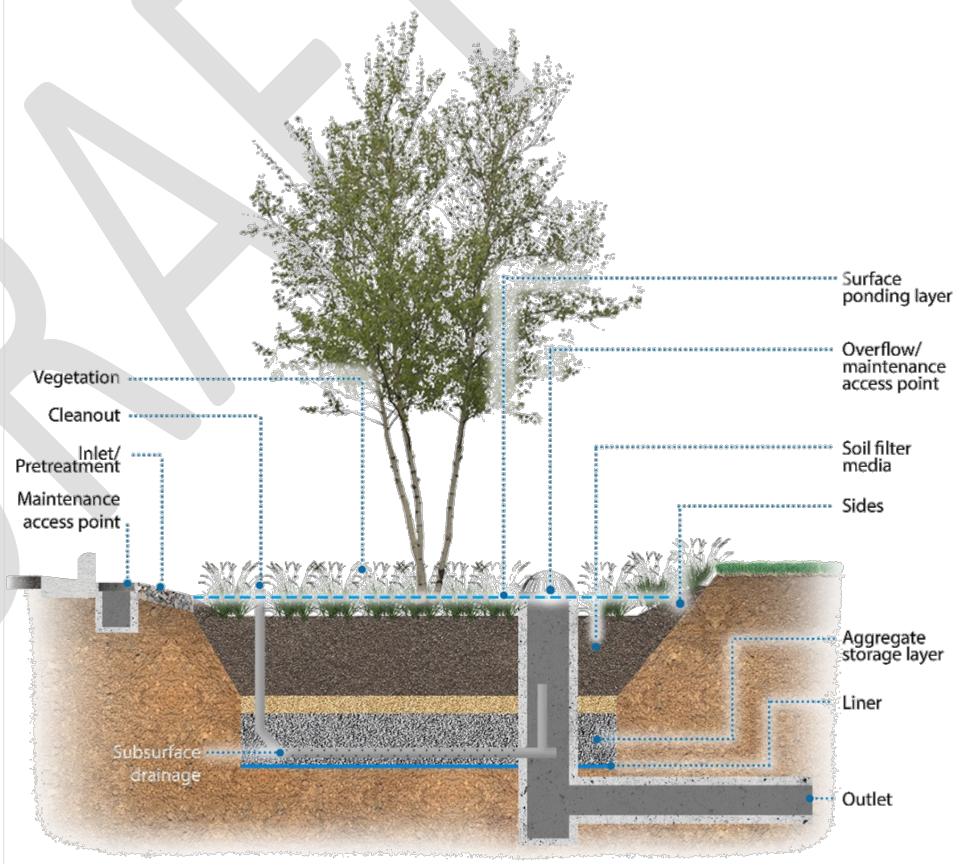

Figure 8-1 Components of a typical bioretention practice

Figure 8-2 Residential Rain Garden

Figure 8-3 CBD Bioretention

Figure 8-4 Commercial Bioretention

Figure 8-5 Bioswale

8.1.2 Configurations

Bioretention practices can be used in a variety of configurations that can adapt to the specific needs of a site. Selection of the most appropriate configuration allows these practices to blend more naturally into the context of the site's surroundings. Below are the basic configurations in which bioretention is commonly applied.

Rain Garden

These practices are usually small and quite simple. They consist of a depressed area with an amended soil or soil filter media capable of infiltrating and filtering stormwater runoff and supporting vegetation. Runoff typically enters the practice from downspouts or via sheet flow across turf grass or small paved areas. Due to the small drainage areas handled by these practices, underdrains are usually not included, and the outlet can be constructed from a low point on one side to direct water to a vegetated area in the direction of an existing conveyance system. These practices should not discharge across paved surfaces such as driveways or sidewalks to prevent the buildup of ice in the winter.

Bioretention Basin

Bioretention basins are larger than rain gardens, but contain many of the same elements. These landscaped depressions or shallow basins typically use a soil filter media to filter and infiltrate runoff that enters through an inlet. Underdrains are incorporated as needed and are connected to an outlet or overflow structure that typically conveys excess flows back into the sewer system. These self-contained, discrete practices may have retaining walls or earthen sloped sides and can support a wide variety of vegetation, including small trees.

Bioswale

A bioswale (or bioretention swale) is a linear practice consisting of a modified swale that uses soil filter media to increase water intake at the soil surface, resulting in improved water quality, reduced runoff volume, and attenuated peak runoff rates for small storms while also providing conveyance of excess runoff. The use of pretreatment control measures such as filter strips or other sediment capturing devices can reduce sediment accumulation in the swale.

Tree-Based Practices

Tree-based practices are variations on the standard bioretention practice designed to infiltrate runoff and support the growth of trees in highly urbanized areas. The two most common types of tree-based practices are suspended pavement systems and tree boxes. Suspended pavement systems consist of some method which provides structural support to overlying pavements while providing ample void space beneath the pavement for root growth. These spaces are filled with lightly compacted soils into which tree roots can extend, thus reducing the footprint of adjacent tree pits or openings. There are a variety of custom systems, usually constructed from concrete, and proprietary modular plastic systems.

Tree boxes are typically pre-cast or cast-in-place concrete structures that can be set adjacent to structural pavements. The boxes are then filled with uncompacted soil filter media, which allow urban trees to thrive by providing space for an extensive root system. Tree boxes often are designed with openings that allow tree roots to extend into adjacent soil. For low to moderate flows, stormwater enters the tree box inlet and filters through the soil. At higher flows, excess stormwater will bypass the tree box and discharge to a downstream curb inlet. Infiltrating tree boxes have an open bottom to allow for infiltration and flow-through tree boxes are completely lined and have a subsurface drainage system to convey flow not absorbed by the soil or taken up by plants to areas appropriate for drainage.

Figure 8-6 Suspended Pavement Sidewalk

Figure 8-7 Suspended Pavement Parking Lot

Figure 8-8 Parking Lot Bioretention

Figure 8-9 Linear Bioretention Along Roadway

Figure 8-10 CBD Curb Extension Bioretention

8.1.3 Landscape Context

The flexible nature of bioretention practices means that they can be used in a variety of situations and combinations to conform to the needs of a site. Whether a site requires stormwater management practices to blend into their surroundings or function as a prominent aesthetic element, bioretention practices can be designed to fit a range of parameters, and are highly suited in the following contexts:

- Along Roadways
- Within Parking Lots
- Adjacent to Buildings
- In Open Spaces

In addition, bioretention practices can also be linked together in a continuous system with other GSI practices to provide treatment for a much larger area.

8.1.4 Site Suitability

Given the wide range of choices and flexibility for the installation of bioretention, it can be difficult to determine which practices and profiles would work well on a selected site. The following table highlights different bioretention configurations and where they are most suitable.

Table 8-1 Bioretention suitability

Configuration	Suitable Sites
Rain Garden	Residential applications, small drainage areas
Bioretention Basin	Adjacent to natural areas, adjacent to parking lots to handle parking lot sheet flow, adjacent to buildings, parking lot islands, larger open areas
Planter Box/ Tree Box	Confined urban sites, parking lot islands, within sidewalks
Raised Planter Box	Adjacent to buildings for roof runoff, sites with shallow groundwater or contaminated soils
Curb Extensions	Along roads
Bioswale	Between rows of parked cars, along roads, adjacent to buildings, adjacent to parking lots
Suspended Pavement	In highly urbanized areas under roads and sidewalks with trees located in the sidewalk, under parking lots with trees located between cars or in small islands

When not to use bioretention:

- Runoff pollutant levels are toxic to plants and animals
- Adequate pretreatment cannot be provided when high sediment loads are present
- Runoff enters practice at high velocities
- Sites with no maintenance access
- Sites that cannot provide an overflow or outlet that does not allow for the practice to drain in 72 hours
- Infiltration practices should not be used when contaminated soils are present or within design setback distances

8.1.5 Function over Time

Because bioretention practices are largely comprised of natural elements, their functions can change over time. Properly designed and constructed practices can flourish and performance can improve over time.

Furthermore, bioretention practices that are appropriately vegetated in a way that increases the aesthetic value of a site have been shown to provide a multitude of benefits. These benefits can increase property values and increase the amount people are willing to pay to live in places with these practices in place (Center for Neighborhood Technology, 2010; Mazzotta, Besedin, & Speers, 2014).

8.2 Bioretention Design Process

Designing bioretention practices requires careful consideration of both existing and proposed site conditions to ensure proper function of the practice. This section provides an overview of the design process with detailed discussions of design and material selection for each component in the following sections. Additionally, Table 8-2 lists chapters that are frequently cross-referenced for any applicable design methods and requirements.

Table 8-2 Cross-referenced chapters

Chapter 2	Regulatory requirements
Chapter 3	Site assessment Conceptual design
Chapter 4	Runoff volume calculations
Chapter 5	Inlet and outlet design Subsurface drainage system design
Chapter 6	Properties of soils and aggregates Soil water calculations Geotechnical requirements

8.2.1 Site Investigation

The ability to retain stormwater on site and to incorporate bioretention practices into the site's stormwater management approach depends on many factors that must be evaluated for each development site. A more detailed explanation on how to conduct a thorough assessment can be found in Chapter 3. As a part of the overall site assessment, the following attributes specifically apply to bioretention practices and should be addressed at a minimum:

- Zoning regulations
- Rights-of-way, road setbacks and property line setbacks
- Utility conflicts
- Buildings and other existing structures
- Surface and subsurface drainage patterns including sewer connections or other available outlets
- Open space availability and suitability
- Natural drainage characteristics and hydrologic flow paths
- Soil suitability for infiltration including the presence of contaminated soils, high groundwater tables or shallow bedrock
- Safety
- Aesthetics

Soil Evaluation

As part of the site investigation, it is imperative to evaluate the existing soils and their suitability for infiltration. Prior to beginning design, Chapter 6 should be reviewed for information on initial site assessments that can be performed to provide general information about the site's soil characteristics, as well as any geotechnical investigations that may be required.

8.2.2 Iterative Design Process

The most cost-effective approach to designing bioretention practices is to use an iterative process that repeats design steps as necessary to ensure the most optimal outcome for the practice (Figure 8-11). Many different processes can be used in designing bioretention practices and are suitable for use as long as the applicable calculations can confirm the sizing of the practice. The recommended process outlined selects the drainage profile (Section 8.3) and sizes the layers, depths and levels (Section 8.4) *prior* to determining storage volumes for the practice. This method accounts for conditions that are typically encountered in Detroit such as native or urban soils that may result in lower infiltration rates; flat topography, which can limit detention storage and the vertical distance between inlet, outlet and overflow elevations; and limited availability of space, which can be common in highly urbanized areas.

Setting the maximum retention depth first based on existing site conditions determines the *minimum* profile depth needed for maximum retention capacity, thus saving money on installation costs. The available vertical space in the profile, along with the applicable dewatering criteria (Section 8.4.1) helps to determine detention capacity of the practice. Once the maximum retention and detention capacities of the vertical profile have been determined, they can be compared to the required storage volumes to see if there is enough area available to layout the practice (Section 8.5), or if additional practices may be required. The components of the vertical profile as well as the location, size and shape of the practice may need to be adjusted as the process progresses to ensure all project goals, objectives and requirements have been met.

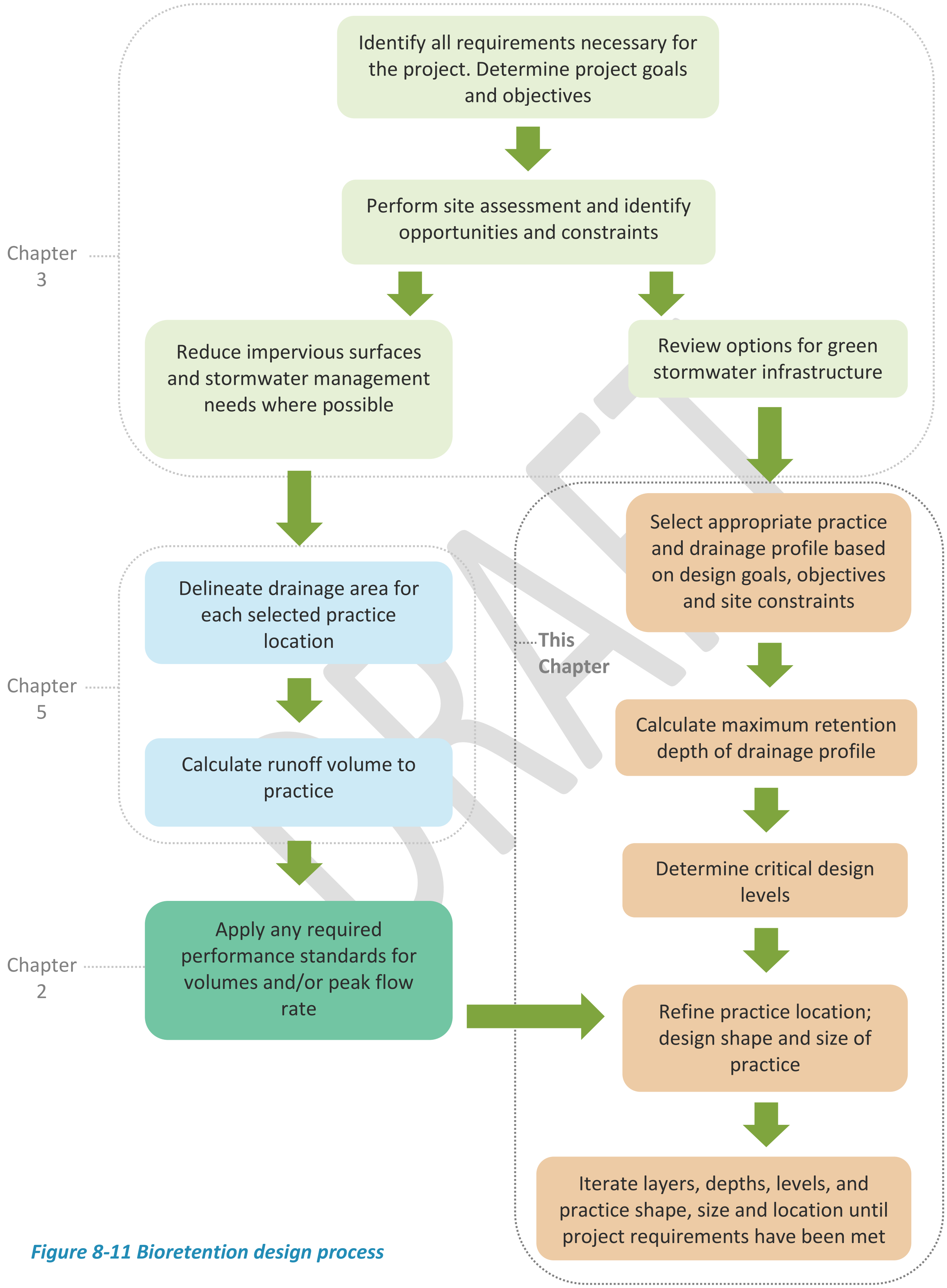

Figure 8-11 Bioretention design process

8.3 Drainage Profiles

The term ‘drainage profile’ refers to how the different vertical layers are designed and assembled to drain and treat stormwater runoff. Given the flexibility of bioretention practices, a variety of drainage profiles could be designed based on design goals and objectives and site constraints. However, there are four basic drainage profiles commonly used:

Conventional

These practices are designed to infiltrate as much water as they can into the surrounding native soils. An underdrain is used with or without an aggregate storage layer to drain away excess water. Water below the underdrain is forced to infiltrate. Conventional drainage profiles have:

- A soil filter media layer typically between 12 and 36 inches deep
- A subsurface drainage system with a minimum of 3 inches of aggregate surrounding each underdrain on all sides
- A flat base below the lowest layer to maximize the area available for infiltration

Conventional drainage profiles work best when the infiltration rate of native soils is less than 0.5 inches/hour, but infiltration is still desired. This profile helps to reestablish natural hydrologic processes, but is typically not recommended for trees and deep-rooted shrubs since their roots can clog underdrains (Figure 8-12).

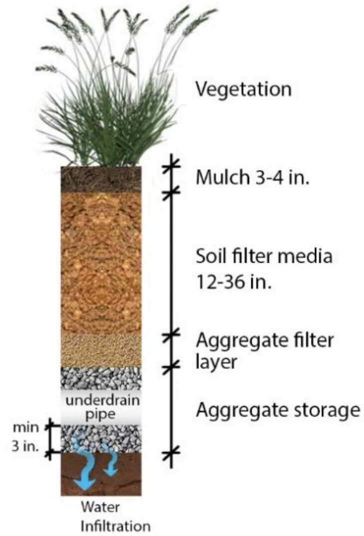

Figure 8-12 Typical conventional drainage profile

Saturated Zone

Practices incorporating a saturated zone include an internal water storage layer at the bottom of the practice while still allowing for infiltration. This saturated storage layer provides a reservoir for vegetation during periods of drought and efficient removal of nitrates from stormwater, which leads to eutrophication and low dissolved oxygen in nitrogen-sensitive watersheds (Brown & Hunt, 2011). When selecting this profile, it is worth noting that the volume within the profile that is still occupied by water after 72 hours cannot be counted toward detention or retention capacity. Saturated zone profiles have:

- A soil filter media layer typically between 12 and 36 inches deep
- A subsurface drainage system fitted with an upturned elbow to create the water storage layer. The underdrains typically reside in an aggregate storage layer for additional storage capacity, but may also be surrounded by a minimum of 3 inches of aggregate on all sides.

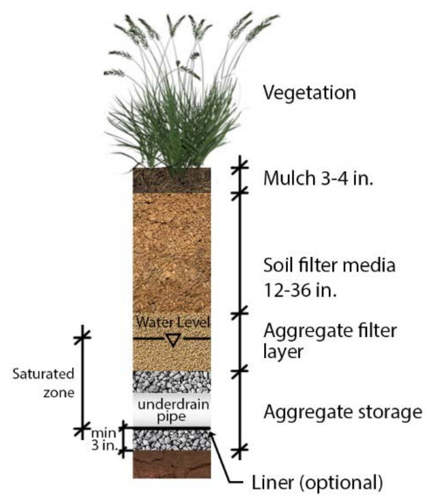

Figure 8-13 Typical saturated zone profile

- An optional impermeable liner or upturned elbow at the end of the subsurface drainage system to retain water in the water storage layer
- A flat base below the lowest layer to increase the area of the saturated zone while maintaining ample room for soils above to drain for plant and biological health

Saturated zone profiles can be implemented to varying degrees. Impermeable liners can be used along the base and sides of the saturated zone for maximum nitrogen fixation, or an upturned elbow can be installed at the end of the subsurface drainage system to create the saturated zone while still allowing for infiltration. They are recommended in nitrogen sensitive watersheds where runoff may contain elevated levels of nitrogen. When a saturated zone is designed to hold water for extended periods of time, they work well in practices containing trees because they provide an additional reservoir of water in times of drought. Furthermore, the saturated zone prevents tree roots from growing into a submerged subsurface drainage system.

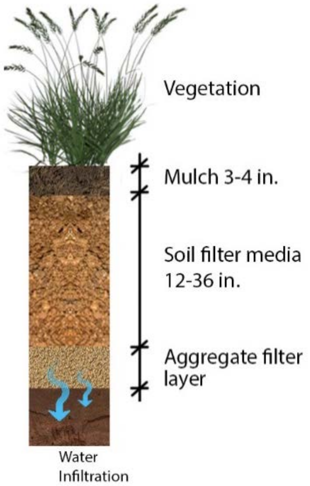

Figure 8-14 Typical pipeless practice profile

Pipeless

Pipeless practices allow for infiltration of all water that enters the practice into surrounding native soils. No underdrains are provided. Pipeless profiles have:

- A soil filter media layer typically between 12 and 36 inches deep
- No subsurface drainage system
- A flat base below the lowest layer to maximize the area available for infiltration

Pipeless profiles work well for small drainage areas where the infiltration rate of the native soil is 0.5 inches/hour or greater. This profile is effective at managing frequent small events and helps to reestablish natural hydrologic processes. Pipeless practices can incorporate trees and shrubs that can withstand periodic inundation.

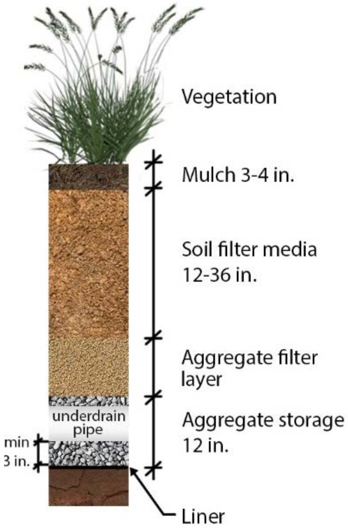

Figure 8-15 Typical sealed practice profile

Sealed

Sealed practices do not allow for infiltration into native soils. All water that does not evaporate or is not used by plants is conveyed through underdrain pipes in the aggregate storage layer. Sealed profiles have:

- A soil filter media layer typically between 12 and 36 inches deep
- A subsurface drainage system within an aggregate storage layer that is typically around 12 inches deep
- An impermeable liner surrounding the base and sides of the practice
- A slightly sloped base that directs water toward the outlet of the practice

Sealed profiles are used when contaminated soils are found or are suspected near the practice, in the presence of karst geology, when the groundwater table is less than 2 feet from the lowest infiltrating level of the practice, when the practice is within 10 feet of a structure or foundation, or in any other situation where infiltration into the native soil is undesirable. These profiles retain the least amount of water of all the bioretention profiles and should only be implemented when site constraints dictate their use.

Profile Selection

Drainage profile selection is governed by site constraints, design goals and objectives and any required performance standards. Most often, site constraints dictate which drainage profile can be used, and therefore, a straightforward process of elimination can help to determine the drainage profile best suited for a given site (Figure 8-16).

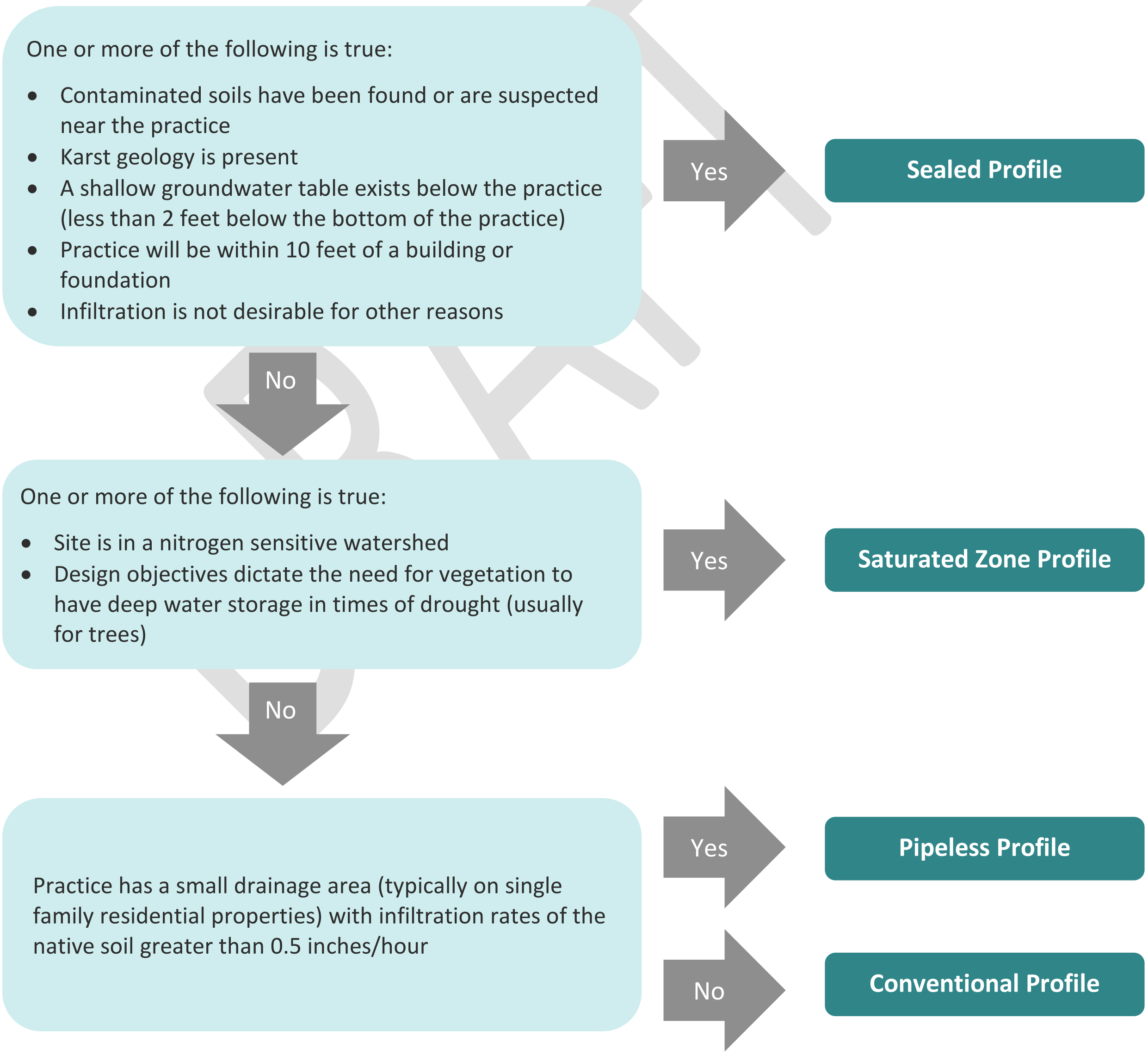

Figure 8-16 Drainage profile selection

8.4 Layers, Depths and Levels

Bioretention practices work by filtering sediments and pollutants from runoff, recharging the groundwater through infiltration and detaining a portion of runoff within the practice. To accomplish these tasks, each layer needs to be designed to the appropriate depth. The design of the layers is dictated by the goals and objectives and site constraints as identified during the site assessment.

The design of the selected drainage profile is an iterative process that works to eventually identify all the following:

- Inlet elevation
- Surface ponding layer depth
- Soil filter media depth and elevation of finished grade
- Aggregate storage layer depth and elevations of layer top and bottom
- Internal water storage layer depth and elevations of layer top and bottom
- Subsurface drainage system invert elevations
- Outlet invert elevations for all connections
- Overflow elevation
- Side heights and slope

8.4.1 Sizing Profile Depth

Prior to sizing the area of the practice and calculating volumes, it is necessary to ensure that there is enough vertical depth to accommodate the selected profile design. The process of sizing the profile depth is outlined below:

1. Determine maximum retention depth
2. Partition maximum retention depth into surface storage depth and subsurface storage depth based on applicable dewatering criteria
3. Convert water depth to corresponding minimum profile layer depths
4. Calculate inlet elevation
5. Set ponding depth
6. Add layer depths below ponding layer to determine the bottom elevation
    - Check against height of groundwater table
7. Determine invert elevation of subsurface drainage
8. Set outlet elevation
    - Check to see that positive drainage can be maintained from outlet to municipal sewer system connection
9. Iterate depths and levels until all design criteria have been met for retention
    - Check to see that profile dewatering criteria is met
10. Adjust ponding depth to accommodate detention storage and set overflow elevation
    - Check to see that elevation is sufficiently below inlet elevation to prevent flooding
    - Check to see that ponding depth meets surface dewatering criteria
11. Iterate depths and levels until all design criteria have been met for detention

Dewatering Criteria

For bioretention practices, there are two dewatering duration criteria that must be met: surface dewatering and profile dewatering. All ponded surface water shall be dewatered within 24 hours following the end of a rain event for purposes of perception and to prevent mosquito breeding. The entire profile of the practice shall be dewatered in 72 hours to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates and maintain adequate soil oxygen levels for healthy soil biota and vegetation.

Key Criteria:
Dewatering Duration

- Surface Dewatering: 24 hours
- Profile Dewatering: 72 hours

Retention Depths

To begin the process of sizing the profile depth, the *maximum retention depth* of the practice needs to be determined. Retention in bioretention practices is governed by the infiltration rate of the native soils and applicable dewatering criteria. The maximum retention depth is the maximum depth of water that can soak into the ground and still meet the allowable dewatering duration criteria (Equation (8.1)). The maximum retention depth that can be stored on the surface can be calculated using Equation (8.2), and the maximum equivalent water depth that can be stored within the subsurface profile can be calculated using Equation (8.3). The following subsections on the different layers of the profile describe how to use the *effective porosity* of the corresponding layer material to convert equivalent water depths into layer depth. Chapter 6, Soil, Aggregate and Water, contains simplified soil water calculations for determining the amount of water that can be stored in a profile. The chapter additionally provides information on effective porosity and other pertinent soil parameters. The equations below are modified to meet the needs of bioretention practices.

Infiltration Rate Safety Factor

0.50

All measured infiltration rates shall be multiplied by a safety factor of 0.5 to obtain the ***design infiltration rate****.*

$$d_{max} = f_c * D_d \tag{8.1}$$

$$d_{sur} = f_c * D_s \tag{8.2}$$

$$d_{sub} = f_c * D_p - d_{sur} \tag{8.3}$$

where d_{max} = maximum retention depth, in
d_{sur} = maximum surface retention depth, in
d_{sub} = maximum subsurface equivalent water retention depth, in
f_c = design infiltration rate, in/hr.
D_d = dewatering duration criteria, hr.
D_s = surface dewatering duration criteria, hr.
D_p = profile dewatering duration criteria, hr.

8.4.2 Surface Ponding Layer

The ability to pond water on the surface of the bioretention practice provides added temporary storage of stormwater, allowing it to filter through the soil media before either infiltrating into the surrounding native soils or exiting the practice through the subsurface drainage system. The depth of the surface ponding layer may vary significantly based on the primary design objectives for the bioretention system, but is typically limited by applicable dewatering duration criteria (Section 8.4.1). Additional considerations for ponding depth may be to meet specific water quality or water quantity requirements, site context and plant health requirements. Additional criteria are associated with bioretention practices incorporated into extended detention basins. Information on detention basin requirements can be found in Chapter 7, Detention Practices.

Typical Ponding Depths

The most common depth of the surface ponding layer in bioretention practices that include vegetation other than turfgrass is 6 inches. This depth is often recommended largely since water depth, frequency of ponding, and duration of both ponding and soil saturation strongly affects both plant growth and the types of plants that can survive in the environment provided. As ponding depth increases, plants' leaves become covered in a fine layer of sediment that can inhibit the plants' ability to perform photosynthesis. At a minimum, ponding depths should be limited to allow for some foliage to extend above the maximum ponding depth.

Site context can also influence the design depth of the surface ponding layer. Bioretention practices that receive only runoff from sidewalks may only have a ponding depth of 2 inches, whereas bioretention practices receiving runoff from roads may pond up to 12 inches. In areas where high pedestrian activity is expected, ponding depth is often limited to 6 inches for safety and aesthetic concerns.

Freeboard Depth

The depth from the top of the ponding layer to the overflow elevation for the practice is called freeboard depth. Depending on the location of the practice within the site and the structures selected to handle overflow, freeboard can be designed either to prevent flooding of adjacent streets or to handle the peak flow from a much larger design storm without causing the practice to overflow.

8.4.3 Mulch

Mulch is a critical component of all bioretention practices vegetated with anything other than turfgrass. In addition to the many benefits mulch provides for plants, it is also responsible for significant filtering functions that help to remove sediments and other pollutants from the stormwater before it enters the soil filter media (Hong, Seagren, & Davis, 2006; Hatt, Fletcher, & Deletic, 2008; Li & Davis, 2008; Stander & Borst, 2010). The flat bottom of the bioretention practice, as well as any other vegetated areas should be covered with mulch when constructed, and annually checked for appropriate mulch depth and condition.

Mulch used in bioretention practices shall meet the following criteria:

- Organic material that is well-aged a minimum of 6 weeks but no longer than twelve months
- Shall not contain deleterious materials including trash, stones, soil, seeds or other plant propagules.
- Grass clippings, leaves or straw shall not be used as mulch due to their elevated levels of weed seed contamination.
- Mulch depth shall be no greater than 4 inches thick. Thicker applications can inhibit proper oxygen and carbon dioxide cycling between the soil and atmosphere, and can also displace surface storage volume.

Many mulch products exist which are suitable for use in bioretention practices and can be selected based on availability, average rate of replacement and material cost. The most commonly used products for mulch are wood mulches and compost mulch. Finely shredded wood products knit together when they are placed and are less likely to float than wood chips or nuggets. Products like compost will need to be replaced more frequently than a wood mulch, and they also tend to leach significant amounts of phosphorous which can be detrimental to freshwater ecosystems. It is recommended that 3 to 4 inches of a triple shredded hardwood bark mulch be used in bioretention practices. Triple shredded mulch is defined as being shredded or processed three separate times and can pass a #3 sieve with average shreds sized between 3/8" and ½" in length.

Benefits of Mulch

- Suppresses unwanted weed growth
- Regulates temperature and moisture in plant root zones
- Source of soil organic matter as it decomposes
- Food source and habitat for beneficial biological organisms
- Filters and collects sediments to maintain infiltrative capacity of soil filter media
- Removes heavy metals from stormwater runoff and breaks down hydrocarbons

Ecosystem services are benefits provided to the human population by functioning ecosystems. Typical ecosystems services provided by bioretention practices include:

- Stormwater management
- Purify water
- Increase biodiversity
- Soil formation
- Aesthetic improvements

8.4.4 Soil Filter Media Layer

Soils are an integral component to the function and success of bioretention practices. Not only do they govern the infiltration and percolation rates of stormwater, but they also filter pollutants, provide a medium and nutrients for plant growth, and are the foundation for a whole host of ecosystem services that are provided by bioretention practices. Soils used in bioretention practices go by a range of names including engineered soil mixes, soil media, blended soils, soil filter media, etc. In most of applications, soil filter media consists of some blend of topsoil, sand and compost in varying proportions, which is why they are commonly referred to as blends, mixes, or media. This section will discuss the ways in which natural soils and engineered mixes can be developed to function as the soil filter media layer.

Calculating Layer Depth

The depth of the soil filter media layer is largely determined by the type of vegetation that is chosen for the practice. While somewhere between 12-36 inches is typically used in most bioretention applications, shallow rooted vegetation like turfgrass can survive in as little as 4 inches of soil filter media. Whereas deep rooted herbaceous vegetation is typically planted in 18-24 inches and tree-based applications are planted in 36-48 inches of soil filter media. Regardless of the recommended ranges for vegetation, it is still necessary to confirm that the equivalent water depth can be contained within the layer. Unless independent testing is conducted on the materials to be used, a value of 0.20 shall be used for the effective porosity of soil filter media.

Effective Porosity of Soil Filter Media

$$n_{eS} = 0.20$$

It is not uncommon to eliminate the aggregate storage layer if the subsurface equivalent water depth (Equation (8.3)) can be managed entirely in the soil filter media layer. One advantage to this, is that plant roots have trouble growing into aggregate, and deeper rooting of vegetation can be achieved without it. Equation (8.4) determines the depth required for the soil filter media layer to hold the subsurface equivalent water depth. If soil depth becomes too large, an aggregate storage layer will be required.

$$d_{soil} = d_{sub} \div \eta_{e\ soil} \tag{8.4}$$

where d_{soil} = required depth of soil filter media layer, in
d_{sub} = maximum subsurface equivalent water retention depth, in
$\eta_{e\ soil}$ = effective porosity of the soil filter media, fraction

Soil Filter Media Requirements

There can be significant variation is the composition of soil filter media due to site conditions, material availability, and design objectives. The requirements and recommended soil filter media mixes below are based on the current research, construction practices and project monitoring in southeast Michigan. However, these may be adjusted in the future as advancements regarding the performance of bioretention soil filter media are made.

The composition and properties of urban soils can have significant spatial variability, even on small sites. This is one of the many reasons that bioretention practices may differ in their functions, design profile, and required soil filter media properties. Therefore, recommendations have been made for mixes that perform well in this area, however other mixes may be used if they meet the requirements in Table 8-3 below. All soil properties must be certified by a qualified soils laboratory with regional State accreditation.

Table 8-3 Soil filter media requirements

Parameter	Range		Testing Method
Particle Size Distribution (Recommended maximum clay content of 15%)	Sieve Size	Percent Passing	ASTM D6913
	2”	100	
	¾”	98-100	
	#4	60-100	
	#10	40-95	
	#40	15-65	
	#200	2-40	
Organic Matter Content	2 – 10% dry weight basis		ASTM D2974, Method C Loss on Ignition Test
Saturated Hydraulic Conductivity	1 – 10 inches/hour		ASTM F1815-11
pH	5.8-7.5		ASTM D4972-13
Soluble Salt Concentration	< 2 mmhos/cm (dS/m)		1:2 soil-water ratio basic soil salinity test
Soil Chemistry	Suitable for growing plants specified		

Designing an Engineered Soil Mix

Blends of any combination of topsoil, sand and organic matter are the most commonly used soil filter media. Mixes usually contain a high proportion of sand to promote infiltration and reduce clogging, and some form of organic matter to support plant growth. Organic matter is typically augmented with the use of compost. However, research has shown that compost can leach significant amounts of nitrogen and phosphorous, which can be detrimental to surface waters (Hunt & Lord, Bioretention performance, design, construction, and maintenance, 2006; Li, Sung, Kim, & Chu, 2010; Roy-Poirier, Champagne, & Filion, 2010; Mullane, et al., 2015). Because of this, it is recommended that bioretention practices located in municipal separate storm sewer system areas limit the amount of compost and phosphorous in the soil filter media. The most common materials used in engineered mixes are discussed below.

Sand

Sand typically makes up the bulk of the mix and must be clean with less than 3% silt and clay content. Most sands used in bioretention soils are medium to coarse sands typically used for masonry and concrete. Examples include ASTM C-33 and MDOT 2NS.

Compost

Compost is well-decomposed stable organic matter that is the result of biological degradation and transformation under conditions designed to promote aerobic decomposition. Compost must meet all the requirements of section ‘Compost as a

Landscape Backfill Mix Component' in the US Composting Council's "Landscape Architecture/Design Specifications for Compost Use" (https://compostingcouncil.org/wp-content/plugins/wp-pdfupload/pdf/32/Landscape-Architecture-Specs.pdf).

Topsoil

For the purposes of this manual, topsoil is defined as the organically enriched, uppermost layer of soil that is commonly identified by its darker color. This layer, also referred to as the A horizon, is the product of natural processes and contains most of all soil biological activity (Weil & Brady, 2017). Artificially mixed products such as sand and compost or peat are not considered topsoil. Topsoil appropriate for use in bioretention practices typically consists of soils conforming to the classification of sandy loam or loamy sand according to the USDA soil texture triangle.

Recommended Mixes for Detroit

The following describes mixes that would work well for different applications in Detroit. While the following mixes are not required to be used, these serve as basic options for a few common applications. Detailed components of each mix are listed in Table 8-4.

Southeast Michigan Standard Mix

This mix is what is commonly used in the region (Carpenter & Hallam, 2010). This mix is best used in situations where water discharging from the practice connects back to a combined sewer due to the elevated levels of nutrients that will leach from the compost.

Tree Mix

This mix is designed for tree pits in commercial areas, tree trenches under suspended pavement or as backfill in soil cell systems like Silva Cell.

Water Quality Mix

A more expensive mix that is best used near sensitive water bodies or in high profile areas where high quality plant material will be used.

Soilless Blend

Soilless blends of sand and compost are now becoming highly recommended as a soil filter media. Due to the relative scarcity of topsoil in the Detroit area, this well blended mixture of a washed, well-graded sand and fine compost serves as another option.

Table 8-4 Composition of recommended soil mixes

	SE MI Standard	Tree Mix	Water Quality Mix	Soilless Blend
Clay & Silt Content	<10% clay	<10% clay	25-40% clay & silt Max clay: 15%	<3% clay & silt
Hydraulic Conductivity	Not Specified	Not Specified	1-4 inches/hour	4 – 10 inches/hour
Nutrient Content	Not Specified	Not Specified	Phosphorous (Mehlich 3): 15-30 mg/kg	Total Nitrogen<1000 mg/kg Phosphorous (Mehlich 3) <80 mg/kg
Organic Matter Content	Not Specified	2% - 4%	2% - 5%	4% - 8%
Suggested Mix Ratios (by volume)	20% Compost 50% Sand 30% Topsoil	10% Compost 30 – 40 % Coarse Sand 50 – 60% Topsoil	50-65% Coarse Sand 25-35% Topsoil 10-15% Compost	60-80% Washed Well-Graded Sand 20-40% Fine Compost

Using Natural Soils

Clean topsoil is quickly becoming a scarce and costly material in southeast Michigan. Reusing existing onsite topsoil helps to save on material and hauling costs and helps to preserve some of the existing structural and biological integrity of the natural soils that are difficult to replicate. Any existing topsoil on site may be reused as the soil filter media if it meets the requirements in Table 8-3 or is amended to meet those requirements.

For projects requiring a Post-Construction Stormwater Management Plan, topsoil shall be tested, at a minimum, for the parameters in Table 8-3 prior to the disturbance of the soil. Topsoil testing may be conducted as part of the required soil investigation (Chapter 6). Given the variability that exists in natural soils, a minimum of two discrete samples must be taken with an additional sample for every 5,000 square feet of topsoil that will be stripped. Depth of topsoil must be determined at every sampling location.

Ways to Reuse Topsoil

- As the soil filter media on smaller projects (amend as necessary)
- As a component of an engineered soil filter media
- For site and turfgrass restoration purposes

8.4.5 Aggregate Storage Layer

An aggregate storage layer may be included to provide an internal water storage layer for retention or detention storage below the soil filter media layer, or to facilitate drainage in low permeability soils or lined practices. There are a few additional considerations that need to be made when using an aggregate storage layer. Since the aggregate is open graded, an aggregate filter layer may be needed to prevent finer particles of the soil filter media from migrating downward. Chapter 6 provides additional guidance on determining if an aggregate filter layer is required. Geotextile filter fabrics shall not be used for this application because they tend to clog or bind as they collect fines, seriously compromising the function of the bioretention practice.

When using an aggregate storage layer with an open-graded aggregate, additional considerations need to be made when selecting vegetation for the practice. The very large pore spaces present in open-graded aggregate layers prevent capillary action from drawing the water upward. This limits the availability of the water stored in this layer to plants and limits how deep the roots of the plants can extend.

Calculating Layer Depth

The depth of the aggregate storage layer is completely dependent on the volume of water that is to be stored there. It is determined by subtracting the water retained in the soil filter media layer (Equation (8.4)) from the subsurface equivalent water retention depth (Equation (8.3)). The remaining equivalent water depth is then converted to layer depth with Equation (8.5). Unless independent testing is conducted on the materials to be used, a value of 0.30 shall be used for the effective porosity of the coarse aggregate.

Effective Porosity of Coarse Aggregate

$$n_{eA} = 0.30$$

$$d_{agg} = (d_{sub} - d_{soil}) * \eta_{agg} \tag{8.5}$$

where d_{agg} = minimum depth of the aggregate layer, in
d_{sub} = maximum subsurface equivalent water retention depth, in
d_{soil} = required depth of soil filter media layer, in
η_{agg} = effective porosity of the aggregate, fraction

Aggregate Filter Layer

Oftentimes, when a soil filter media is placed directly on top of an open-graded aggregate storage layer, the finer particles of the soil filter media migrate downward into the much larger pore spaces of the aggregate. To prevent this from happening, both layers either need to be designed such that migration between the two layers does not happen, or a filter layer (also referred to as choker layer or transition layer) needs to be placed in between. Geotextile fabrics are not suitable for this application due to their high susceptibility for clogging from the fines present in the soil filter media. Including an additional aggregate layer with particle sizes that are in between that of the soil filter media layer and the aggregate storage layer is the best way to ensure permeability and prevent migration of fines moving into or through adjacent layers. Typically, this layer is designed with a thickness between 4 and 6 inches. Additional aggregate filter layer sizing criteria are included in Chapter 6, Soil, Aggregates and Water.

Materials

Aggregates used in the construction of the aggregate storage layer should consist of locally available, naturally occurring, clean aggregate. Recycled crushed concrete should not be used due to its alkalinity and propensity to change the pH of water passing through the practice. The following aggregates meet the requirements for the aggregate storage layer:

- sand –clean medium concrete sand (ASTM C-33 or MDOT 2MS)
- pea gravel –double washed fine aggregate (MDOT 34G, AASHTO #78 or #8) having a maximum loss by wash (passing #200 sieve) of less than 0.5%
- coarse aggregate –MDOT 6A/6AA coarse aggregate or AASHTO #57 with a maximum loss by wash (passing #200 sieve) of less than 1%

Suitable materials for the aggregate filter layer are based on the properties of the soil filter media and the aggregate selected for the storage layer. Selection of materials shall be based on the sizing criteria found in Chapter 6, Soil, Aggregates and Water.

8.4.6 Liners

When the selected drainage profile or site conditions require the separation of materials, a variety of liners can be used. Liners can either be impermeable to impede the flow of water or permeable to allow water to pass through. Additional information regarding the selection of liners can be found in Chapter 6, Soil, Aggregates and Water.

Impermeable Liners

Impermeable liners are used as part of the sealed drainage profile when adverse site conditions are present or when close proximity to buildings precludes the infiltration of water into native soils. They are also used to a lesser degree in the saturated zone drainage profile to create the underground zone of saturation.

Impermeable liners can be constructed from suitable clay, geomembranes, or designed as concrete retaining walls with a sealed bottom. All seals between liner sheets, or seals at pipe connections or perforations shall be watertight.

Permeable Liners

Permeable liners may be used when there is a desire to prevent materials from mixing but infiltration is still desired and feasible. However, use of permeable liners is strongly discouraged in situations where fines may lead to clogging and adversely affect infiltration rates in the practice.

8.4.7 Layers Relative to Groundwater

When groundwater tables are too close to the bottom of an infiltrating practice, it can have potential negative effects. High seasonal or permanent groundwater tables can reduce the infiltration capacity of the practice, leach pollutants and nutrients into the groundwater, or even lower groundwater levels and discharge groundwater back into the municipal sewer system.

Therefore, the distance from the elevation of the lowest infiltrating layer of the bioretention practice to the seasonal high groundwater level or bedrock is recommended to be four feet. Two feet is the minimum allowable distance, but may reduce the performance of the practice.

Key Criteria:

Depth to Groundwater (from bottom of practice)

- Suggested Depth: 4 feet
- Minimum Depth: 2 feet

8.5 Layout

The layout for bioretention practices needs to account for a variety of factors during the design process to ensure that sufficient space is available. The section below details the most common layout considerations.

8.5.1 Location

Placement and integration with other site elements should be incorporated as early in the conceptual design as possible to minimize any space conflicts and maximize the most suitable locations. Factors to consider when locating bioretention within the site layout include most suitable soils for infiltration practices, available space, elevation and location of proposed practice regarding how water will be routed to the bioretention practice, and maintenance access. The following is a list of settings where bioretention can be incorporated to meet more than one project-level or watershed-scale objective:

- Parking lot islands
- Other common landscaped areas
- Open space edges
- Within rights-of-way along roads

Listed below are additional requirements that must be addressed when finalizing the location of bioretention practices.

Setback Requirements

Bioretention practices are typically designed to collect and infiltrate stormwater runoff. It is important to maintain minimum separation distances between infiltrating practices and building foundations to prevent saturated soils that can negatively affect their performance, or from water supply wells to minimize the chance of contamination. Refer to Table 8-5 (SEMCOG, 2008).

Table 8-5 Setback Distances

Setback from	Minimum distance (feet)
Property line	10
Building foundation[1]	10
Private well	50
Public water supply well[2]	50
Septic system drainfield[3]	100

1. Minimum with slopes directed away from building
2. At least 200 feet from Type I or IIa wells, 75 feet from Type IIb and III wells (MDEQ Safe Drinking Water Act, PA 399)
3. 50 feet for septic systems with a design flow of less than 1,000 gallons per day

Pedestrian Areas

Additional care shall be taken when proposing bioretention practices near pedestrian corridors and access points. Areas adjacent to vehicle exit zones shall allow for safe exit from a vehicle onto a level surface without risking a large drop or stepping into water. Bioretention designs shall include a 1.5- to 2-foot wide safety zone with a maximum longitudinal slope of 2.0% between any sloped or uneven surfaces and pedestrian areas, such as sidewalks and curbside parking. Furthermore, passenger vehicle doors shall be allowed to fully open without striking vegetation or any raised structures associated with the practice.

Key Criteria:

Pedestrian Areas

Ensure that the location of bioretention practices does not prohibit ADA compliance

Maintenance Access

Regular maintenance is a key component to ensuring the long-term functionality of bioretention practices. Typical maintenance activities such as sediment removal, weeding, watering, trash removal, and inspection of structures, requires considerations for access during the design. A full list of maintenance activities is presented in Section 8.11. It is recommended that this section be reviewed to ensure that proper access is provided for their execution.

Safety

Bioretention practices do not typically pose any major safety concerns. Considerations such as ponding depth and pest control are handled by required dewatering criteria (Section 8.4). Inlets are required to be designed with minimal flow velocities and sized to prevent flooding of adjacent pavement (Section 8.6.1), and overflow structures prevent the practice from flooding the surrounding area (Section 8.7.2). Practices located adjacent to heavily trafficked pedestrian areas may want to incorporate low fencing or vertical curbing (Figure 8-17), especially when the practice includes vertical walls.

Figure 8-17 Vertical curbing adjacent to sidewalk

8.5.2 Sizing the Practice

> **Flow Configurations**
>
> **Online/through-flow**
>
> Stormwater flows into the practice until capacity is reached. Excess volume either flows out the downstream end toward the surface storm drainage system or bypasses the practice. Excess water shall not back up into a street.
>
> **Offline**
>
> All stormwater directed toward the practice flows in and is detained until maximum ponding depth is reached. A raised overflow structure manages additional flows above maximum ponding depth and connects into an underground stormwater conveyance system. Overflow structures shall be designed to handle the 10-year peak flow (see Chapter 4).

Once suitable locations have been determined and drainage profiles have been selected and adequately sized, the area of the practice footprint can be determined. Like all the steps involved in designing bioretention practices, determining the area is an iterative process and may require further modifications to the type of drainage profile, layer depths and levels. The steps below detail a simplified approach to determining the area of the practice to meet the applicable regulatory requirements. Sizing the practice using Darcy's Law (Chapter 6) is also an acceptable method, but is not required.

- Using Chapter 2, Regulatory Requirements, determine which regulatory requirements are applicable to the practice.
- Delineate the drainage area contributing to the practice.
- Calculate the runoff volume desired to be permanently retained, infiltrated onsite, using the methods discussed in Chapter 4, Hydrologic Procedures.
- Divide the calculated runoff volume by the equivalent water depth stored in the practice including any surface storage calculated in Section 8.3 to determine the surface area of the practice required to infiltrate the runoff volume. This area corresponds to the **lowest horizontal surface area for infiltration** of the practice (Figure 8-18).
- Starting from the bottom layer, use the batter (side slopes) of each layer above the lowest practice layer to determine the area of the practice footprint at the surface (Figure 8-18).
- Ensure that all setbacks, maintenance access requirements and safety considerations are still being met.

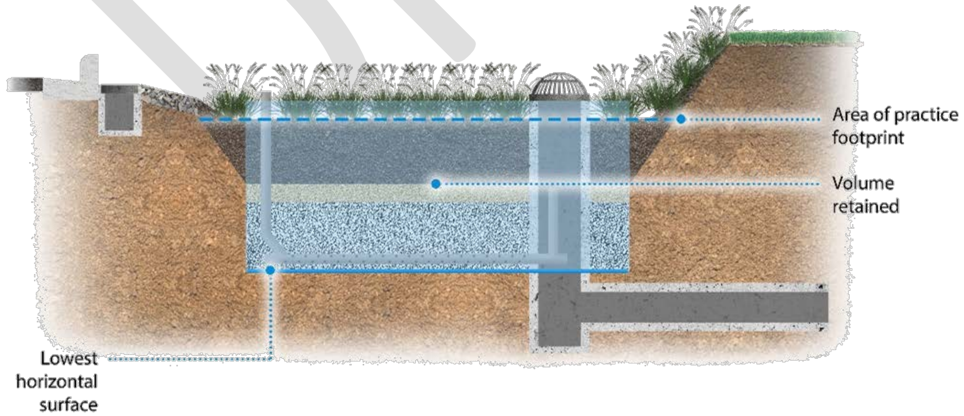

Figure 8-18 Bioretention cell with a conventional drainage profile and offline flow configuration

Lowest Horizontal Surface Area for Infiltration

Bioretention practices that infiltrate into native soils are designed with a flat bottom to maximize the amount of infiltration. In practices with vertical sides, the lowest horizontal surface area corresponds to the area of the practice footprint. However, construction of practices without retaining walls frequently requires excavation of existing soils at a sufficient angle to prevent sides from caving in. This results in sloped sides where bioretention storage and filter media materials are in direct contact with native soils. While some water may be lost through the sides of the practice, this surface area decreases over time as the saturated water level decreases. Therefore, calculating retention volumes using the area of the practice footprint are likely to overestimate the actual retention volumes. Without additional modeling capable of calculating more accurate retention volumes based on the three-dimensional characteristics of the practice, infiltration into non-horizontal areas of the native subgrade will be neglected.

8.5.3 Shape of Practice

The shape of a bioretention practice is governed by the area required for retention volumes, locations of inlets and outlets, and the surrounding site context. Bioretention practices located between roads and sidewalks are typically long and narrow and may use vertical walls instead of sloped sides to increase the area available for infiltration. Practices located in areas with ample space, however, may take on more organic forms with undulating side slopes to blend more naturally into the surrounding environment.

The width of the practice footprint shall be no less than 2 feet; anything less than this is difficult to construct. A minimum width of 4 feet is recommended to provide more flexibility in vegetation selection. It is recommended that the practice be no wider than 50 feet when construction access is available from both sides. This dimension ensures that average construction excavators can reach the entire width of the practice from the top of the slope to prevent unnecessary soil compaction.

Bioretention Sides

The sides of the bioretention practice form the transition from the practice footprint up to existing grade. Sides can be constructed from vertical concrete walls, stacked stone, segmental block retaining walls, or simply graded earth with a slope back to existing grade. When using a graded side slope, the steepness of the slope is determined by several factors. Typically, maximum slopes on bioretention sides with turf grass are 4H:1V to facilitate mowing. Gentler slopes may be used as space allows, or when bioretention practices are more than 3 to 4 feet deep to allow a person or animal to climb back out. Steeper slopes are used when vegetation other than grass will be used to stabilize the slope. 3H:1V is the typical maximum slope in this situation, but steeper slopes may be used in very shallow practices or if it can be determined that the side will be stable. More information on the construction of sides in deep practices is included in Chapter 7, Detention Practices.

8.6 Inlet and Pretreatment

Inlets control the volume and flow rate of water that can enter the bioretention practice. Correct selection and design of inlet structures is highly dependent upon the site layout, the volume and velocity of stormwater runoff to be managed by the inlet, and the volume of debris and sediment the stormwater is carrying. Oftentimes, pretreatment devices and energy dissipation to prevent erosion and scour can be designed as components of a single inlet structure as opposed to separate elements.

8.6.1 Inlet Design

The type and spacing of the selected inlet dictates how flow will be distributed across the practice footprint, if scour will occur, and how sediments will be managed. Procedures for calculating inlet spacing and capacity are covered in Chapter 5, Drainage Conveyance. While bioretention inlets can be designed in many ways, there are three types that are most commonly used:

- Curb cut (concentrated surface flow)
- Flow spreader (distributed surface flow)
- Piped inlet and inlet structure

Curb Cuts

When bioretention practices are incorporated directly adjacent to highly impervious areas, such as parking lots and in road rights-of-way, curb cuts are often the most cost-efficient means of directing surface runoff into the practice. Curb cuts can be designed in a variety of ways to ensure that water enters the practice without causing excessive scour in the practice or ponding water in travel lanes.

In situations where water velocities are high, curb cut openings can be designed in series to allow a portion of the water to bypass the first inlet and travel into the next. This scenario is common in online practices where total interception of larger storms is not desired, but is not suitable for curb cuts located in sag points.

Figure 8-20 Curb cut

Figure 8-19 Curb cut bypassing CB

Designs have the following recommendations:

- The opening should be a minimum of 12 inches wide at the base to prevent clogging and to provide dispersed flow.
- The curb cut can have vertical sides or have chamfered sides at 45 degrees.
- The gutter and bottom of the curb cut shall be sloped toward the stormwater practice to ensure low flows do not bypass the inlet.
- A minimum 2-inch drop in grade between the curb cut entry point and the finished grade of the practice shall be provided.
- The curb cut must pass the design storm flow for the practice without causing water backup into travel lanes.

Flow Spreaders

Larger bioretention practices and bioswales can oftentimes be designed with a curbless inlet that allows discharge to be directed to the bioretention practice as surface sheet flow. When space allows, gravel fringes between pavement and grassed filter strips can act as an inlet, pretreatment, and inlet protection all in one by dissipating energy and evenly distributing the flow.

Gravel fringes should consist of a 4-inch layer of MDOT 4AA stone (underlain by geotextile filter fabric) extending 2 to 3 feet from the pavement edge, where space allows (Figure 8-21). Ensure that a suitable subbase for the road extends under the gravel fringe to ensure proper drainage and to protect the structural integrity of the pavement section.

Figure 8-21 Gravel fringe flow spreader

Filter strips should ideally be sodded and graded at 4H:1V slopes or flatter (3H:1V maximum for vegetated slopes that will not require mowing). Any slopes that convey flow should be routinely inspected for rill erosion, which can contribute excessive sediment to the bioretention area and often represents the most common maintenance issue (Wardynski & Hunt, 2012). Take care to prevent flow from concentrating between parking lot curb stops/blocks.

Piped Inlets and Inlet Structures

Piped inlets are typically used when the bioretention practice is located away from the impervious surface area it manages. These can be used when there may be utility conflicts in the right-of-way, pedestrian conflicts that may compromise safety, or when space is not available directly adjacent to the contributing impervious surfaces. Calculations necessary to correctly size piped inlet are included as part of Chapter 5.

Figure 8-22 Piped inlet

Figure 8-23 Upturned inlet

Inlet structures, including drop structures and diversion structures can be used to regulate the amount of flow that enters the bioretention practices as well as control sediments. Information regarding structure sizing is included as part of Chapter 5.

Bioretention areas that treat runoff from residential roofs or other cleaner (low sediment and debris yield) surfaces might not require pretreatment for trash or sediment but should include energy dissipation to the extent practicable. Energy dissipation can be provided by upturning inflow pipes so that they bubble up diffusely onto a rock apron (Figure 8-23)

8.6.2 Pretreatment

Pretreatment is designed to capture floatable trash, gross solids and excessive sediment before it reaches the practice footprint to prolong the functional life of the soil filter media. Bioretention practices operate more effectively, have a longer functional life, and require less maintenance when pretreatment is provided to remove sediment ahead of, or at the inlet to, the practice (SEMCOG, 2008). Unless a bioretention practice is treating runoff from impervious surfaces that have little to no sediment load, such as roofs, some form of pretreatment is required at all inlets.

Pretreatment that passes sheet flow over vegetated filter strips or through grassed swales is preferred because in addition to settling solids of incoming stormwater runoff, they also act as energy dissipaters and encourage additional infiltration. However, when these practices are not feasible, a variety of other pretreatment options including rock forebays, sumped inlets, sediment/grit chambers, or manufactured water quality devices may be used. It is highly encouraged that additional source control measures such as sweeping paved surfaces (driveways, parking lots, etc.) and catch basin cleaning be implemented within the contributing drainage area of the practice to reduce sediment loads entering the practice whenever possible.

Figure 8-24 Vegetated filter strip leading to bioretention

Vegetated Filter Strip

Filter strips are bands of dense, permanent vegetation with a uniform slope, primarily designed to provide water quality pretreatment between a runoff source (i.e., impervious area) and the receiving bioretention practice (Figure 8-24). The inflow to a filter strip must be conveyed as sheet flow. Typically, this is accomplished by installing a level spreader system immediately upstream of the filter strip. Filter strips are well suited for treating runoff from roads, parking lots, and disconnected downspouts. Filter strips provide water quality improvement primarily through vegetative filtering (i.e., increased settling of sediment-bound pollutants due to the shallowness of overland flow and flow attenuation provided by flow path roughness) and infiltration. Reductions in runoff volume from small storms can be achieved if the soils are sufficiently pervious, sheet flow is maintained along the length and width of the strip, and contact time is long enough for infiltration to occur.

Key Criteria:
Maximum Flow Velocities

- Grassed Surfaces: 3 ft./sec
- Mulched Surfaces: 1 ft./sec

8.6.3 Inlet Protection

Inlet protection is required to minimize damage whenever concentrated flows enter the practice. Design of these measures will vary depending on site layout, but they all are required to dissipate excess energy, act as a level spreader, and stabilize the inlet point. Rock pads and rock forebays are the most common types of inlet protection. The specified size, shape, and weight of the rock used are a function of the velocity of the water and the geometry of the area to be protected. A geotextile fabric must be placed below the rock to prevent excessive settling. Armored inlets (Figure 8-25) and drop structures can be used where space allows. Calculations for sizing the aggregate and determining the dimensions of the rock splash pad can be found in Chapter 5, Drainage Conveyance.


Figure 8-25 Armored inlet

8.7 Outlet and Overflow

The outlet for a bioretention practice shall be designed to meet the hydraulic requirements and to minimize vandalism, clogging from trash and debris, and the need for frequent maintenance. Access for maintenance shall be provided. The outlet shall connect to a stormwater conveyance system, either underground or at the surface, depending on the application. Bioretention outlet design should consider the characteristics of the contributing drainage area and the anticipated quantity and type of trash and debris.

8.7.1 Outlet Design

The surface outlet may be a simple overflow structure designed to pass runoff more than the water quality design event (Figure 8-26), or may be a multi-stage outlet that allows the bioretention practice to also provide extended detention for larger runoff events (**Error! Reference source not found.**). A primary consideration when establishing the height of the surface outlet(s) is that the design should allow the surface water in a basic bioretention practice to fully drain within 72 hours. Bioretention practices designed for extended detention may have longer drain times, refer to Chapter 6, Soil, Aggregate and Water.


Figure 8-26 Permeable aggregate parking lot outlet structure

The overflow may be designed in a variety of configurations. The two most common approaches include a catch basin overflow located within the practice that is raised above the practice footprint surface elevation, and a curb cut exit routing water back to the gutter pan in online practices.

Overflow outlets must meet the following minimum requirements:

- Structures shall have a minimum 2-foot diameter. Structures with internal weirs, baffles, weir plates, underdrains with upturned elbows, backflow preventers, valves or other flow control devices shall be a minimum 4-foot diameter.
- Structures located within the influence of pavement shall be constructed of concrete.
- Castings shall be iron.
- Castings shall be a domed beehive or angled grate.

8.7.2 Overflow Structures

For all bioretention practices that are designed as offline practices to receive flows that exceed the design capacity of the practice, an overflow structure will be required. Overflow structures can be a simple pipe or weir in the side of the practice to convey excess flow volumes downstream toward the surface stormwater conveyance system. Typically, in places like Detroit where the topography is relatively flat, an overflow structure will need to connect into a nearby underground stormwater conveyance system.

It is recommended that overflow structures be located as close to the inlet as possible to minimize the flow path for exceedance flows to reduce the risk of scouring.

8.8 Subsurface Drainage

A subsurface drainage system consisting of a network of underdrains is required for all bioretention practices that either include impermeable liners, are placed on top of soils having infiltration rates less than 0.5 inches per hour, or when practices are within 50 feet of a steep (greater than 20%) or sensitive slope. All underdrains must meet the following minimum requirements:

- All pipes and fittings shall be Schedule 40 or SDR 35 smooth wall PVC pipe. Corrugated HDPE will not be allowed.
- All pipes shall have a minimum diameter of 6 inches.
- Bend fittings shall not exceed an angle of 45 degrees.
- Riser pipes, cleanouts, and all piping not located within the bioretention practice, or within 5 feet of a structure connection shall be solid walled pipe.
- Underdrain laterals shall be perforated with a minimum of 3 rows of 3/8-inch diameter perforations around the circumference of the pipe. Perforations shall be placed 6 inches on center within each row for the entire length of the pipe.
- Underdrain laterals shall be installed either within an aggregate layer (Figure 8-27), or bedded and covered within a gravel envelope (minimum 3” bedding, minimum 3” cover) to prevent migration of soil into the underdrain.
- To prevent clogging, underdrain pipes shall not be wrapped with a geotextile.
- A cleanout location shall be provided at the terminal ends of each underdrain.

8.8.1 Subsurface Drainage Design

The subsurface drainage configuration greatly affects water movement through the profile of a bioretention practice, the amount of water held for retention, and hydrologic and water quality performance. Detailed design calculations for the subsurface drainage system are included in Chapter 5, Drainage Conveyance.

Conventional Drainage Profile

Conventionally drained practices feature an underdrain that freely drains and outlets near the elevation of the subgrade (Figure 8-27).

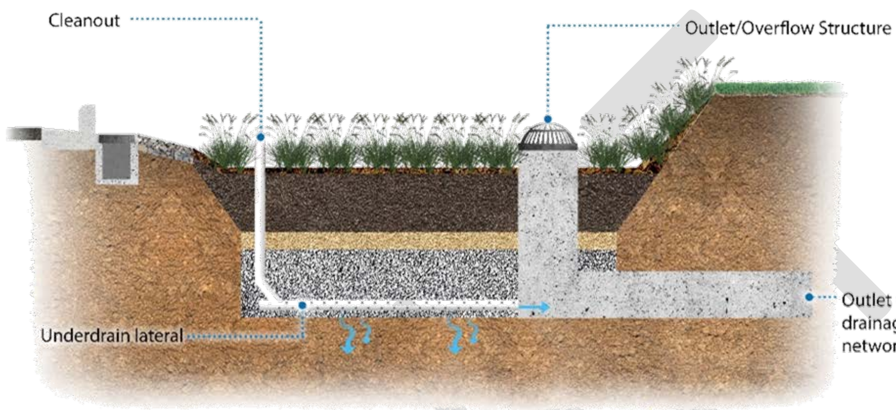

Figure 8-27 Bioretention cell with a conventional drainage profile

Saturated Zone Drainage Profile

Infiltration and pollutant load reduction can be further enhanced by upturning the underdrain to create an internal zone of saturation (Brown & Hunt, 2011). This saturated zone enhances exfiltration into underlying soils while maintaining aerobic soil conditions in the plant rooting zone above. It is most convenient to upturn the underdrain inside the outlet or overflow structure using a tee-connection (Figure 8-29).

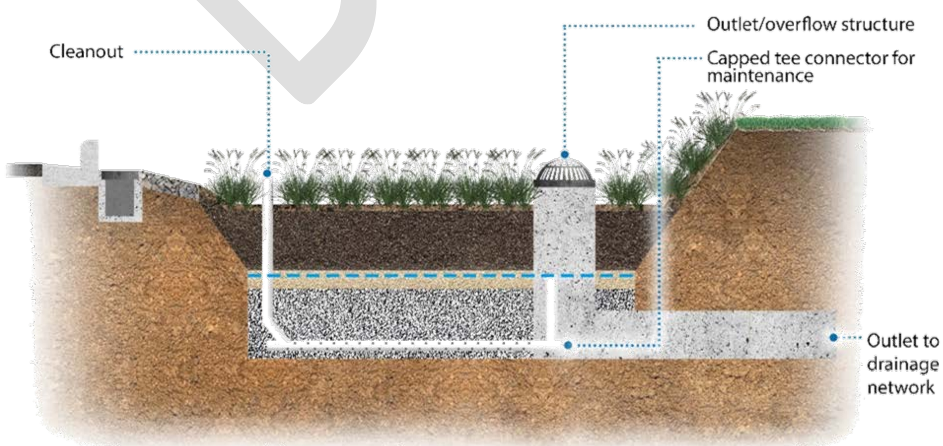

Figure 8-29 Bioretention cell with a saturated zone drainage profile

Rocky Mount, North Carolina. Source: NCSU BAE

Figure 8-28 Upturned underdrain with capped tee connection

By placing the underdrains at the bottom of the aggregate storage layer and controlling the depth of the saturated zone with an upturned elbow, the height of the vertical pipe inside the outlet structure can be adjusted based on the performance of the practice overtime. Furthermore, the capped tee-connection allows easy access to the subsurface drainage system for inspection and maintenance (Figure 8-28). To provide an aerobic root zone and to reduce mobilization of previously captured pollutants, the top of the saturated zone should be at least 18 inches below the surface of the soil media filter layer (Hunt, Davis, & Traver, 2012). Additionally, it should extend above the aggregate storage layer into at least the aggregate filter layer to promote upward movement of the water through the soil profile to be accessed by plant roots. Any volume of water that remains within the saturated zone after 72 hours cannot be counted for retention or detention storage.

Sealed Drainage Profile

When the bioretention practice is lined with an impermeable layer, the underdrains are placed at the bottom of the practice (Figure 8-30). Typically, the bottom of the practice slopes slightly toward the underdrain to remove excess water. However, the underdrains must also have a minimum slope of 0.5% toward the outlet to ensure the practice drains in 72 hours.

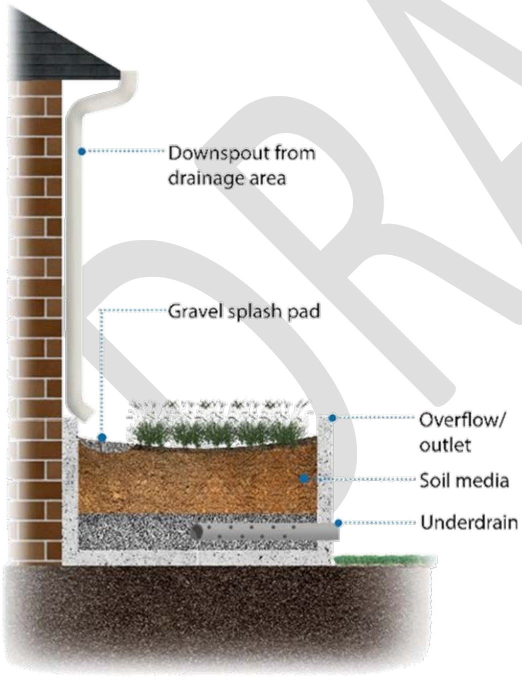

Figure 8-30 Bioretention cell with a sealed drainage profile

8.8.2 Retention Volume

The volume of stormwater that can be retained in a bioretention practice changes dramatically when underdrains are employed. In these practices, the volume retained is only the storage volume that is located below the subsurface drainage system outlet which is capable of entirely draining in 72 hours or less (Figure 8-31).

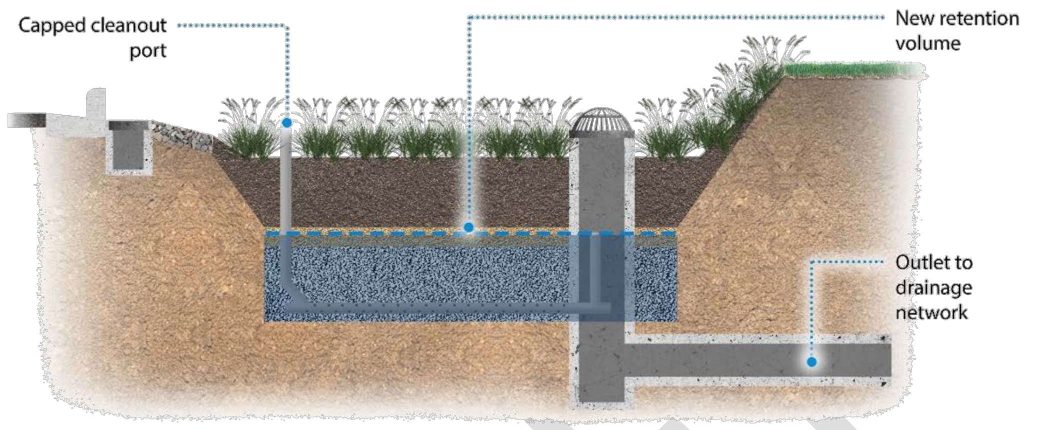

Figure 8-31 Cross sectional area of volume retained with an underdrain

$$V_R = V_A * n_{eA} + V_S * n_{eS} \quad (8.6)$$

where V_R = retention volume, ft^3
V_A = volume of aggregate below subsurface drainage system outlet, ft^3
V_S = volume of soil filter media below subsurface drainage system outlet, ft^3
n_{eA} = effective porosity of aggregate, fraction
n_{eS} = effective porosity of soil filter media, fraction

8.9 Vegetation

Many of the benefits provided by bioretention are a direct result of proper vegetation selection. Healthy, thriving vegetation can help to remove and break down a wide variety of pollutants, provide erosion control and bank protection, maintain and increase infiltration rates, soak up water, and provide a multitude of social, environmental and economic benefits. Vegetated practices can also aid in satisfying any landscaping, open space, or tree canopy requirements. For vegetated practices to thrive, proper plant selection is crucial. Urban environments, coupled with the challenging conditions created by the bioretention practices themselves provide a considerable number of stressors that can affect the success of the vegetation, all of which must be accounted for in the plant selection process. Improper plant selection can result in plant death, significant weed growth, reduced soil structure, and increased levels of soil compaction, just to name a few.

Figure 8-32 Residential rain garden

Because the conditions within a bioretention practice can vary so greatly, it can be quite difficult to find a variety of plants that can thrive in *all* the conditions that are present. So, in many situations, it may be useful to consider different zones within the practice and select vegetation that would be best adapted for those specific environments.

8.9.1 Species Selection

There are a few common attributes that apply to vegetation selected for bioretention regardless of the specific site conditions it faces. Plants with extensive or deep root systems help to improve the practice's function over time. Roots can physically, chemically and biologically modify surrounding soil to aid the creation of macropores and stabilization of soil structure, all of which increase rates of infiltration (Weil and Brady, 2017). Other attributes pertinent to vegetation selection such as their ability to withstand regional climate extremes, fast growth rates and the vegetation's origin are discussed below.

Figure 8-33 Switchgrass: a common deep-rooted native grass

Plant Hardiness

Extreme cold temperatures are a major determining factor of whether or not plant material will survive in a given location. According to the 2012 USDA Plant Zone Hardiness Map, Detroit falls within zone 6b, meaning that all plant material selected should be able to withstand an average annual extreme minimum temperature of -5 degrees to 0 degrees Fahrenheit. This information is usually listed in plant catalogs or on plant tags in nurseries.

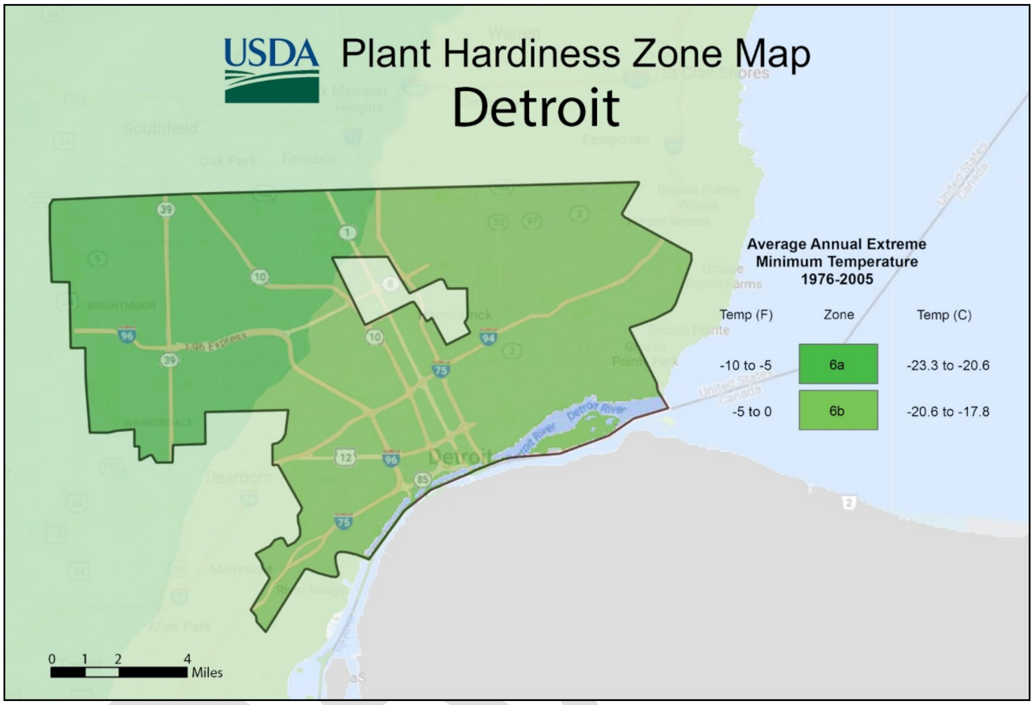

Figure 8-34 USDA plant hardiness zones for the city of Detroit

Growth Rates

It is important to establish a healthy plant community as quickly as possible to combat invasive perennial species, summer annuals, and reduce the occurrence of erosion. Plants with moderate to fast growth rates and deep, extensive root systems are highly advantageous when it comes to achieving this, especially in challenging environments. Additionally, vegetation with deep and extensive root systems aid in the formation of soil structure, which can be especially important in engineered bioretention soils where nearly all soil structure has likely been destroyed during the mixing process.

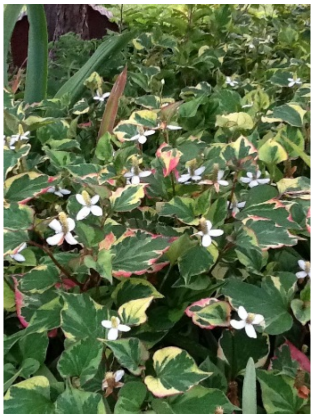

Figure 8-35 Chameleon plant is a highly aggressive groundcover not suitable for natural environments

Prior to selecting plant species with aggressive growth rates, it is extremely important to fully understand the potential impacts they may have on surrounding environments. Aggressive plants are well suited for use in highly urbanized environments. However, those that can self-propagate far beyond the reaches of where they were initially planted may not be suitable for natural environments where they may escape and displace valuable native plant communities. Even native species can become overly dominant when artificially introduced, so it is important to understand how aggressive plants propagate and ensure that the proper environment and barriers are present prior to selecting these species.

Natives vs. Non-Natives

When selecting vegetation for bioretention practices, the vegetation's origin should hinge on the larger goals of the project. Vegetation should complement the surrounding landscape and should be capable of performing well given the site's current, not historic, ecological conditions. Many colorful and attractive non-native plant species are well-adapted to the harsh conditions bioretention practices create. Additionally, there is little to no research to prove that native plants have an advantage to survival over their non-native counterparts in bioretention applications (Coletta, 2014). Which is why it is incredibly important to select vegetation based on the objectives and constraints presented by each project. Chapter 3, Site Design and Stormwater Management, provides greater detail on identifying specific site constraints, determining design goals and how to use design principles to meet these goals.

8.9.2 Site Conditions

As previously mentioned, urban environments provide many challenges for vegetation. The presence of closely spaced buildings can create wind tunnels, temperature extremes and even stark contrasts in the amount of available sunlight within a few feet. Large swaths of impervious surfaces further increase temperatures and can transport many urban pollutants such as sediments, PAHs, metals, and deicing salts to name a few. To further complicate matters, GSI practices create conditions of inundation followed by drought, supply stormwater with high concentrations of pollutants, especially following small storm events, and often are placed in very confined spaces. All of this takes place on top of urban soils which are frequently highly disturbed, compacted and low in nutrients. Additional attributes must be considered during the site assessment (Chapter 3, Site Design and Stormwater Management) to properly account for these conditions.

8.9.3 Hydrologic Zones

Plant material should generally be tolerant of summer drought, extreme heat and ponding fluctuations with standing water for up to 24 hours. However, most vegetated stormwater management practices can be divided into three separate hydrologic zones in which the prevalent soil moisture conditions present can have distinct variations. It then becomes much easier to select plants that have stronger drought tolerance for upland areas and plants with high inundation tolerances for the lowest areas.

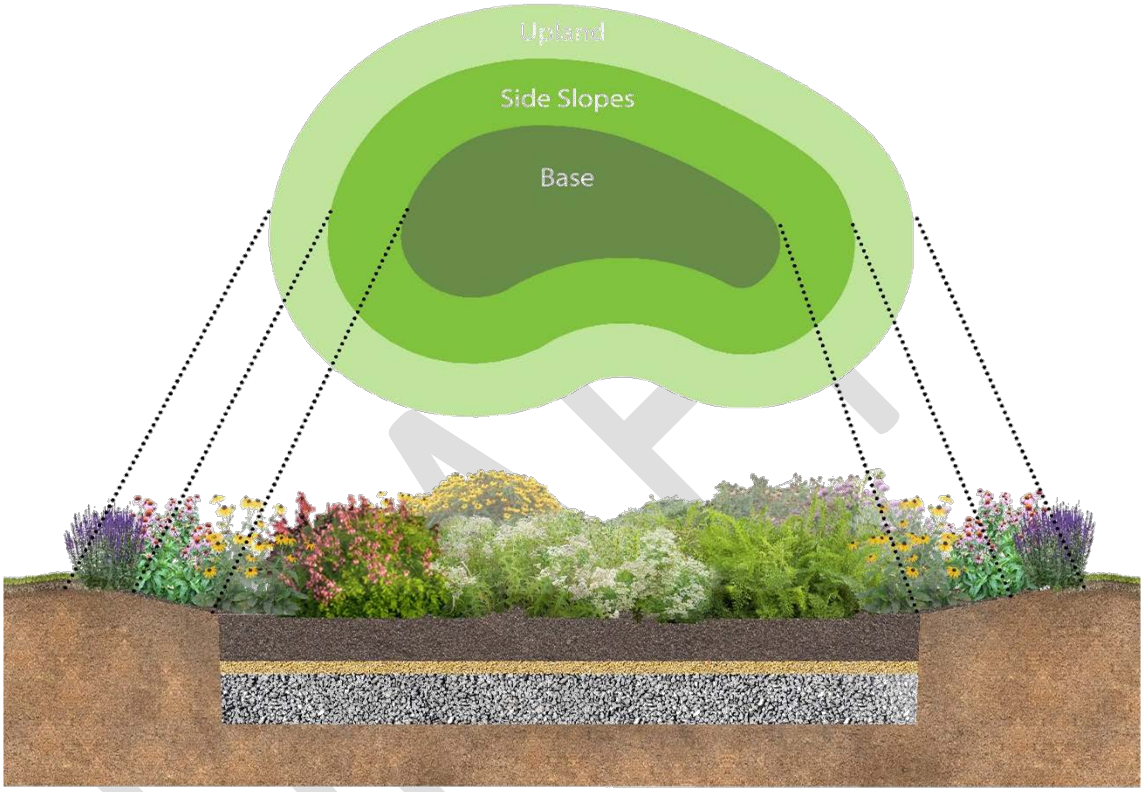

Figure 8-36 Hydrologic Zones of a Typical Vegetated GSI Practice

Upland

These areas of the practice are situated at the top of the slope and may also act as a buffer zone between the practice and any surrounding turfgrass or hardscape. These areas will almost never see flooding, and any surface runoff directed to the practice will rarely provide any significant available water to the roots of these plants. Therefore, it is important to select plant material that is highly drought tolerant.

Side Slopes

Depending on the depth of the practice, this area will likely see the highest fluctuation in soil moisture content. Since this is the area where ponding occurs, frequent to infrequent flooding must be tolerated by plant species, and they will commonly be listed with moisture requirements as moist to dry soils.

Base

This area is located at the lowest elevation of the practice where ponding and saturated soils will frequently occur. Plants that thrive in wet meadow conditions where soils are moist to saturated will do best here, and should be tolerant of saturated soils for up to 72 hours.

8.9.4 Special Considerations

Inlets and Sediment Forebays

Figure 8-37 Sediment damage on plants

These areas face especially harsh conditions and may not be suitable for plants. Sediment deposition on leaves and on the soil surface can reduce photosynthesis, negatively affect seed germination, and can lead to depressed plant productivity and even death (Shaw & Schmidt, 2003). Plants may be successfully placed adjacent to these areas, however, and should be robust and vigorous growers with dense root systems to stabilize any potentially erosive areas. Plant material nearest any inlets should also be tolerant of higher levels of pollutants, such as high nutrients, salt, metals, and hydrocarbons. Ensure that plant material in high sediment areas is tall enough to provide leafy vegetation above the maximum ponding depth to allow for continued photosynthesis.

Salt Tolerance

There are two types of salt tolerance that need to be considered with plants. The first is aerial spray tolerance and mostly applies to trees and shrubs that have above ground growth present during the winter and early spring. This is especially important in right-of-way applications where vehicles can splash nearby vegetation or send clouds of salty mist into the air. The second is soil salt tolerance. This mostly applies in right-of-way applications, near sidewalks, in parking lots, or any application where the practice is receiving stormwater from an area that has been salted. Plants that germinate later in the spring have an advantage over early season plants since spring rains help to reduce the concentrations of salt in the soil before growth begins. Resources for plants with salt tolerance can be found here:

- Oakland County, MI, Salt-Tolerant Plant List https://www.oakgov.com/msu/Documents/publications/oc0257_salt_tolerant.pdf
- Christensen's Plant Center Salt Tolerant Landscape Plants http://www.christensensplantcenter.com/docs/SaltTolerance.pdf
- Ottawa Conservation District – Michigan Native Plants http://www.ottawacd.org/pdfs/a_few_michigan_native_plants.pdf

8.9.5 Recommended Plant Lists

There are a variety of resources available for the selection of plant material suitable for use in vegetated stormwater management practices in Southeast Michigan. Below are a few examples of resources available online:

- SEMCOG manual, Appendix C
  http://semcog.org/Reports/LID/index.html
- Landscaping for Water Quality
  www.michigan.gov/documents/deq/ess-nps-lwq-overview_209328_7.pdf
- Plants for Stormwater Design: Species Selection for the Upper Midwest
  https://www.pca.state.mn.us/water/plants-stormwater-design
- Philadelphia Green Stormwater Infrastructure Landscape Design Guidebook
  http://documents.philadelphiawater.org/gsi/GSI_Landscape_Guidebook.pdf

Resources for invasive plants in the Midwest and in Michigan can be found here:

- MDNR Michigan Invasive Species List
  http://www.michigan.gov/invasives/0,5664,7-324-68002_71240---,00.html
- A Field Identification Guide to Invasive Plants in Michigan's Natural Communities
  http://mnfi.anr.msu.edu/invasive-species/InvasivePlantsFieldGuide.pdf
- Midwest Invasive Species Information Network
  http://www.misin.msu.edu/

8.10 Construction Considerations

Construction techniques and sequencing are critical to bioretention practice performance. Failure of improperly constructed practices can be minimized by hiring contractors with specialized training for landscape-based stormwater practices or previous experience in successfully constructing bioretention practices. Effective communication with the contractor and inspection during key steps are also necessary for successful construction. Emphasizing the following points will help ensure successful installation of bioretention practices.

8.10.1 Construction Phasing

The use of vegetation and soil-based GSI practices requires careful attention to construction staging and phasing. Protection of soils from compaction and disturbance during site preparation and construction, soil amendment, the installation of soil filter media, and the timing, placement and techniques used in planting, all affect the ultimate efficacy of bioretention practices. Therefore, a construction and phasing plan must be included as part of the construction documents for all vegetated GSI practices to ensure proper installation, function, and treatment.

Construct Bioretention After Site Is Stabilized

Bioretention practices should not be constructed until all other areas within the drainage area have been constructed and stabilized to avoid heavy sediment runoff from prematurely clogging the filter media.

Minimize and Mitigate Compaction

Compaction of underlying soils can decrease infiltration rates resulting in poor drainage from the practice. Whenever possible, bioretention practices should be designed to allow for excavation of the practice to take place without needing heavy equipment to enter the footprint of the practice (Section 8.5.3). It is recommended that infiltration tests be performed on the subgrade prior to placing any of the bioretention layers to confirm native soil infiltration rates. If measured infiltration rates are below what was used during design, or if compaction was unavoidable during excavation, Chapter 6, Soil, Aggregates and Water, provides a variety of methods for mitigating the effects of compaction.

Figure 8-38 Scarifying soil with a toothed excavator bucket

Prior to placing any of the bioretention layers, the subgrade surface must be properly prepared. The surface of the subgrade should be flat (except for bioswales) and final elevations verified with survey equipment. The surface of the subgrade soil should have a rough texture, commonly created with a toothed excavator bucket (Figure 8-38). Smooth, shiny surfaces are indicative of smearing, which creates a sealed surface that is relatively impenetrable by water, and not desired.

Soil Filter Media Placement

It is important to ensure that the soil media is consistent with specifications before installation—media that is too sandy will not provide adequate treatment, whereas media that is too fine might not drain in adequate time (Carpenter & Hallam, 2010). To field-verify the texture of soil media, moisten the soil and form into a 1-inch ball. Drop the ball from 1 foot onto the open palm of the hand. The ball should break apart on impact, indicating that it is a sandy soil. When rubbed between the fingers, the moist soil should also leave a thin layer of mud residue on the skin, indicating that fines are present in the mix. Soil media should also contain a small amount of plant-based organic matter evenly distributed throughout the mix— the organic matter should not smell like manure. Note: These inspection techniques are intended for field verification and do not substitute for laboratory soil test results.

Upon installation of the soil filter media, some level of settling will occur. It is important to account for this settling during design by specifying a method of construction to ensure that final design elevations are maintained following installation. The most common methods for achieving this are as follows:

- Place soil filter media with as little compaction as possible and allow the soil to settle on its own over time. Designs should include an additional 5-10% above the desired design depth of the soil filter media layer to account for this settling.

- Place soil filter media with as little compaction as possible and saturate the soil with water following placement. All depressions should be filled with additional soil filter media and saturated again with water. This process should be repeated until the soil filter media layer is flat and at the desired design elevation following saturation.
- Place soil in 12-inch lifts and gently compact following the placement of each lift. Lifts are generally compacted to a recommended 85% maximum density.

8.10.2 Verify Critical Design Elevations

It is important to verify intended design elevations for layer depths, inlet, outlet and overflow elevations, pipe slopes, and invert elevations to ensure that the appropriate volumes can be captured by the bioretention practice.

Contractors who are unfamiliar with construction of bioretention practices may try to minimize surface ponding by installing the outlet elevation too low. Practices that do not provide their intended capacity of surface storage will overflow more often than intended. Therefore, it is critical to check the design surface ponding depth has been provided and the bed of the cell has been uniformly graded—this can be performed by simply verifying the overflow/bypass elevation of the practice relative to the average elevation of the mulch bed surface. An average depth must be measured because the height of the outlet structure relative to adjacent ground surface is not a reliable indicator of average ponding depth (Wardynski & Hunt, 2012).

8.11 Operation and Maintenance

Bioretention practices require regular plant, soil, and mulch layer maintenance to ensure optimal infiltration, storage, and pollutant-removal capabilities. Table 8-6 provides a detailed list of maintenance activities. Bioretention practices differ in design and function in several critical ways from traditional landscaped areas. Specialized training in management of landscape-based stormwater practices is recommended for maintenance staff, landscaping contractors or volunteers. This training may be available through local or state landscape contractor's associations, master gardener courses, or other conservation district or park district workshops. In general, bioretention maintenance procedures consist of the following:

Erosion control - Inspect flow entrances, ponding area, and surface overflow areas periodically, and replace soil, plant material, or mulch layer if erosion has occurred. Also inspect side slopes that receive sheet flow to identify and correct areas that are eroding from excessive concentrated flow velocities. Properly designed, constructed and maintained facilities with appropriate flow velocities should not have erosion problems except perhaps in extreme events. If erosion problems occur, the following must be reassessed: (1) flow velocities and gradients within the practice, and (2) flow dissipation and erosion protection strategies in the pretreatment area and flow entrance. If excess sediment is deposited in the bioretention practice, immediately determine the source within the contributing area, stabilize, and remove excess surface deposits. Any exposed soil in the tributary drainage area should be permanently stabilized with grass, rock, or other erosion-resistant materials.

Inlet - The inlet of the bioretention area should be inspected semiannually (early spring and late fall), or as needed in response to large storm events, to check for sediment accumulation and erosion. Any accumulated sediment that impedes flow into the bioretention area should be removed and properly disposed.

Overflow and underdrains - Sediment accumulation in the overflow device or underdrain pipes can cause prolonged ponding and potential flooding. Excess ponding can have adverse effects on vegetation and vector control. Overflow and subsurface drainage systems should be inspected quarterly, or as needed in response to large storm events, to remove sediment and prevent mulch or debris accumulation around the overflow. The subsurface drainage system should be designed so that it can be flushed and cleaned as needed. If water is ponded in the bioretention area for more than 72 hours, the subsurface drainage system should be checked by flushing with clean water until proper infiltration is restored.

Plant material - Depending on aesthetic requirements, occasional pruning and removal of dead plant material might be necessary. Replace all dead plants and, if specific plants have a high mortality rate, assess the cause and, if necessary, replace with more appropriate species. Periodic weeding is necessary until plants are established. The weeding schedule can become less frequent if the appropriate plant species and planting density have been used and, as a result, undesirable plants are excluded.

Nutrient and pesticides - The soil mix and plants are selected for optimal fertility, plant establishment, and growth. Nutrient and pesticide inputs should not be required and can degrade the pollutant processing capability of the bioretention area and contribute pollutant loads to receiving waters. By design, bioretention areas are in areas where phosphorous and nitrogen levels are often elevated. If in question, have the soil analyzed for fertility.

Mulch - Replace mulch annually in bioretention areas where sediment, heavy metal or other pollutant deposition is a concern (e.g., contributing areas that include industrial and auto dealer/repair parking lots and roads). In areas where excess pollutant accumulation is not a concern, add mulch as needed to maintain a 4-inch depth (DO NOT OVERMULCH). Mulch should be replaced every 2 to 5 years.

Soil - Soil mixes for bioretention areas are designed to maintain long-term fertility and pollutant processing capability. Estimates from metal attenuation research suggest that metal accumulation should not present an environmental concern for at least 20 years in bioretention practices (Davis, Sholouhian, Sharma, Minami, & Winogradoff, 2003; Morgan, Paus, Hozalski, & Gulliver, 2011). Replacing mulch in bioretention areas where heavy metal deposition is likely, provides an additional level of protection for prolonged performance. If in question, have the soil analyzed for fertility and pollutant levels and consult MDEQ and local regulations for disposal protocols.

Watering - Plants must be selected to be drought tolerant and not require watering after establishment (2 to 3 years). Watering may be prudent during prolonged dry periods after plants are established.

Table 8-6 Bioretention inspection and maintenance tasks

Task	Frequency	Indicator maintenance is needed	Maintenance Notes
Catchment inspection	Weekly or biweekly with routine property maintenance	Excessive sediment, trash, or debris accumulation on the surface of bioretention	Permanently stabilize any exposed soil and remove any accumulated sediment. Adjacent pervious areas might need to be regraded.
Inlet inspection	Weekly or biweekly with routine property maintenance	Internal erosion or excessive sediment, trash, or debris accumulation	Check for sediment accumulation to ensure that flow into the bioretention is as designed. Remove any accumulated sediment.
Litter and leaf litter rermoval	Weekly or biweekly with routine property maintenance	Accumulation of litter and leafy debris in the bioretention area	Litter and leaves should be removed to reduce the risk of outlet clogging, reduce nutrient inputs to the bioretention area and to improve facility aesthetics.
Pruning	Prune dead and broken branches annually and deciduous shrubs every 3-5 years	Overwgrown vegetation that interferes with access, lines of sight or safety	Nutrients in runoff often cause bioretention vegetation to flourish.
Mowing	2-12 times/year	Overwgrown vegetation that interferes with access, lines of sight or safety	Frequency depends on location and desired aesthetic appeal
Mulch removal and replacement	1 time/2-3 years	Less than 4 inches of mulch remains on the surface	Mulch accumulation reduces available surface water storage volume. Removing decomposed mulch also increases surface infiltration rate of fill soil. Remove decomposed fraction and top off with fresh mulch to a total depth of 4 inches
Temporary watering	1 time/2-3 days for first 1-2 months, sporatically after establishment	Until established and during sever droughts	Watering after the initial year might be required.
Fertilization	1 time initially	Upon planting	One-time spot fertilization for first year vegetation.
Remove and replace dead plants	1 time/year	Dead plants	Plant die-off tends to be highest during the first year (commonly 10% or greater). Survival rates increase with time.
Outlet inspection	Monthly	Erosion at outlet	Remove any accumulated mulch or sediment.
Miscellaneous upkeep	12 times/year	Tasks include trash collection, plant health, spot weeding, removing invasive species, and removing mulch from the overflow device.	

8.12 Design Checklist

The design of bioretention practices typically involves many iterations, design of several individual components, and frequent modifications during the design process. To ensure that the practice has been properly designed, the following checklist shall be used upon completion of the design, but before construction drawings have been finalized. This checklist can then be included as part of the Post-Construction Stormwater Management Plan along with any required calculations to document the design.

Table 8-7 Design checklist

Description	Design Requirement	Recommended Values	Design Value	Units
Treatment				
Drainage area tributary to practice				acres
Peak flow rate requirement	See Chapter 2 for applicable performance standards			cft/sec
Retention volume requirement	See Chapter 2 for applicable performance standards			cft
Design Flows and Volumes				
Minor design storm volume entering practice				
Minor design storm peak flow rate entering system				
Major design storm volume entering practice				
Major design storm peak flow rate entering system				
Drainage Profile				
Design infiltration rate of native soils		Greater than 0.5 in/hr. for pipeless profiles		in/hr.
Drainage profile type				
Depth of soil filter media layer		4 inches for turfgrass 18-24 inches for deep rooted herbaceous vegetation 36-48 inches for trees		in
Depth of aggregate filter layer				in

Description	Design Requirement	Recommended Values	Design Value	Units
Depth of aggregate storage layer	Minimum of 4 inches (if applicable)	4 to 6 inches		in
Liner type (permeable, impermeable, none)				
Surface dewatering duration	Maximum of 24 hours			hr.
Profile dewatering duration	Maximum of 72 hours			hr.
Practice Elevations				
Inlet elevation				ft.
Top of inlet protection elevation	Minimum of 2 inches below inlet elevation			ft.
Overflow rim elevation				ft.
Maximum surface ponding elevation		0.50 feet above top of soil filter media elevation		ft.
Top of sides elevation				ft.
Top of soil filter media elevation				ft.
Average underdrain invert elevation				ft.
Upturned underdrain elevation				ft.
Outlet invert elevation				ft.
Bottom of practice elevation	Minimum of 2 feet above groundwater table elevation (not applicable to practices that do not infiltrate)	Minimum of 4 feet above groundwater table elevation (not applicable to practices that do not infiltrate)		ft.
Groundwater table elevation below practice				ft.
Maximum wall height (side slope is vertical)				ft.
Layout				
Area of practice footprint (lowest surface elevation)				sft
Area of practice including slopes				sft

Description	Design Requirement	Recommended Values	Design Value	Units
Area of lowest horizontal surface area for infiltration				sft
Minimum practice width	Minimum of 2 feet	Minimum of 4 feet		ft.
Maximum practice width		Maximum of 50 feet when construction access is available from both sides (25 feet for access from one side)		ft.
Bottom slope	Flat, except for bioswales			H:V
Average slope of sides		4H:1V to 6H:1V		H:V
Maximum side slope		4H:1V for turfgrass 3H:1V for other vegetation		H:V
Distance to nearest property line	Minimum of 10 feet			ft.
Distance to nearest building foundation	Minimum of 10 feet (not applicable to practices that do not infiltrate)			ft.
Distance to nearest private well (where applicable)	Minimum of 50 feet (not applicable to practices that do not infiltrate)			ft.
Distance to nearest water supply well (where applicable)	Minimum of 50 feet (not applicable to practices that do not infiltrate)			ft.
Pedestrian areas, maintenance access and safety have been addressed	Confirm all applicable ADA requirements have been met, confirm maintenance procedures can be performed			
Inlet Design				
Inlet type				
Width of opening		Minimum of 12 inches for curb cuts		in
Method of pretreatment	Required unless runoff is from impervious surfaces with little to no sediment load (i.e. roofs)			

Description	Design Requirement	Recommended Values	Design Value	Units
Method of inlet protection	Required when incoming flow velocities are greater than 3 ft./sec on grassed surfaces and 1 ft./sec on mulched surfaces			
Peak flow velocity over inlet protection (minor storm)	3 ft./sec when discharging to grassed surfaces 1 ft./ sec when discharging to mulched surfaces			ft./sec
Peak flow velocity over inlet protection (major storm)	3 ft./sec when discharging to grassed surfaces 1 ft./ sec when discharging to mulched surfaces			ft./sec
Subsurface Drainage System Design				
Minimum underdrain pipe diameter	Minimum of 6 inches			in
Minimum dimension of aggregate surrounding pipes	Minimum of 3 inches			in
Subsurface drainage system capacity				cft/sec
Overflow Design				
Overflow type				
Overflow weir length				ft.
Overflow capacity				cft/sec
Outlet Design				
Outlet Type				
Outlet pipe size				in
Outlet capacity				cft/sec
Vegetation				
Type of vegetation				
Depth of mulch				in

8.13 References

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9. Infiltration Practices

Infiltration practices may recharge groundwater and help maintain stream baseflow; preserve and enhance vegetation; reduce pollutant loads transported to receiving waters; and reduce the total runoff volume, peak flows and temperature. Infiltration practices may take many different forms including basins, beds, vaults, swales, trenches and dry wells. This chapter discusses different configurations and the respective design standards for each. Also discussed are methods to increase infiltration, pretreatment for sediment, freezing weather considerations, and groundwater contamination concerns.

Contents

9.1 Introduction

Infiltration practices are designed to encourage percolation and groundwater recharge, which in turn, reduces stormwater runoff from the site along with many of its negative side effects. Infiltration practices can remove pollutants from stormwater through the interaction of the chemical, physical, and biological processes that occur between soils and water. These processes help to filter out sediments, and dissolve and capture excess nutrients from infiltrating stormwater.

Infiltration practices come in a variety of configurations that can be fit onto most sites. However, many of the same benefits derived from infiltration practices are also offered by bioretention and permeable pavement practices (Chapter 8 and Chapter 10, respectively). Because these two GSI practice types offer a multitude of additional benefits above and beyond the practices discussed in this chapter, they should be considered for use first.

Figure 9-1 Infiltration trench installed adjacent to drive lane

Advantages

- Very effective at removing pollutants and reducing runoff volumes
- Underground practices may be placed under site elements like parking lots or recreational areas when space is limited
- Large volumes of water can be stored underground to eliminate many of the safety concerns associated with detention ponds

Limitations

- Do not offer the same aesthetic and ecological benefits as bioretention practices
- Many practices require large areas to achieve the level of infiltration that is desired
- High sensitivity to sediment clogging, which requires increased levels of maintenance
- Underground practices can be more costly and difficult to install than surface practices

9.1.1 Configurations

Infiltration practices can be used in a variety of configurations, and similar configurations often go by a variety of different names, creating unnecessary confusion. The section below categorizes the typical configurations of infiltration practices according to the size required for the infiltration footprint, and if water is stored on the surface or below ground as it infiltrates into the soil below.

Infiltration Basin

Infiltration basins are large shallow surface impoundments designed to infiltrate stormwater over a level and uncompacted surface. Infiltration basins use existing soil and vegetation on the surface. An infiltration basin may resemble a detention basin in that it typically includes an inlet, forebay, storage zone, outlet works and an emergency overflow. A forebay is commonly used for sediment pretreatment with an infiltration basin. Smaller versions of infiltration basins may resemble bioretention practices, i.e. they may be shallow depressed areas with vegetation (often turfgrass).

Figure 9-2 Infiltration Basin

Infiltration Bed

Infiltration beds are large shallow areas where the water is stored underground. The purpose of the infiltration bed is to provide a place to temporarily store water underground while waiting to infiltrate. The water is typically stored in an aggregate media, e.g. an open graded aggregate reservoir. A variety of materials may be used to help temporarily store the water underground such as soil; sand; expanded shale, clay and slate (ESCS); asphalt treated permeable bases (ATPB), and a variety of manufactured support systems. Infiltration beds are commonly used under bioretention and porous pavement systems. Water typically reaches the infiltration bed by filtering through a soil or permeable pavement layer first, hence a separate sediment pretreatment device is not commonly used.

Figure 9-3 Infiltration Bed

Infiltration Vault

Infiltration vaults are large underground storage cells intended to infiltration water. The added storage provides extended time for infiltration to occur. Vaults are commonly constructed of perforated pipes, arched chambers (e.g. plastic or CMP), or concrete structures. Infiltration vaults are commonly designed for both infiltration and detention storage. These types of systems are often placed under parking lots as a space savings technique. Water is commonly piped into the vaults and requires some form of sediment pretreatment (e.g. a sump or manufactured treatment device).

Figure 9-4 Infiltration Vault

Infiltration Swale

Infiltration swales are a linear application of an infiltration basin. The swales are vegetated and designed to infiltrate stormwater through the bottom as well as convey the water down the swale. Sediment removal is commonly achieved by filtration through the soil and plants, thus trapping the sediment on the surface of the swale. Sediment traps may be placed in the swale to further improve sediment removal.

Infiltration Trench

Infiltration trenches are a linear application used to store and infiltrate water below grade. Infiltration trenches often include a perforated pipe system surrounded by an open graded aggregate. Infiltration trenches may receive stormwater from surface runoff that flows down into the trench, or stormwater may be conveyed through a perforated pipe and allowed to exfiltrate. Infiltration trenches may be constructed under parking lots and roadways. Pretreatment is commonly achieved using a vegetated buffer strip or vegetated swale for cases where the water enters from the top. When water is piped into an infiltration trench from anything other than a roof or other sediment free impervious surface, a sump or manufactured treatment device is required.

Figure 9-5 Infiltration Trench

Dry Well

Dry wells are small discrete storage devices (typically a cylinder or a tank) that are designed to drain by infiltrating into the native soil. For residential applications, a common dry well approach is to route a roof drain to a buried 50 gallon perforated plastic container surrounded by an open graded aggregate. Along roadways and in parking lots, dry wells can be installed in place of a catch basin. In these cases, dry wells are commonly a perforated manhole or catch basin surrounded by open graded aggregate. These may also be referred to as *leaching basins*.

Figure 9-6 Dry Well

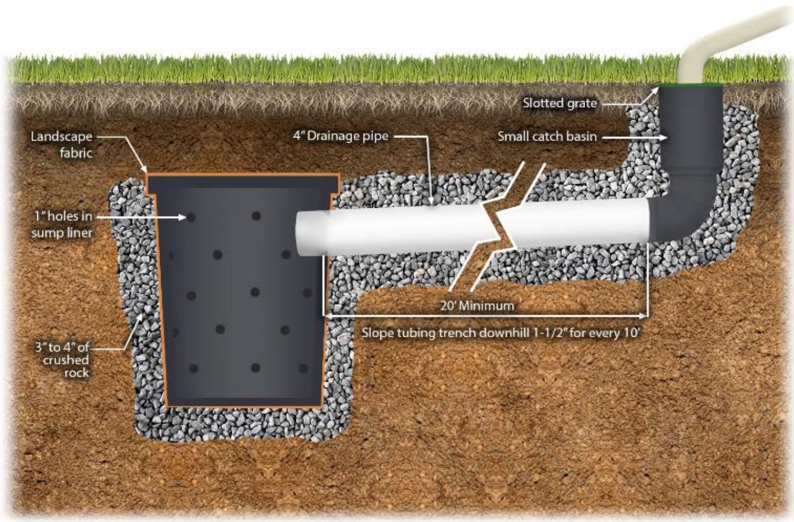

Figure 9-7 Dry well attached to roof downspout

9.1.2 Site Suitability

Given the very wide range of choices when it comes to incorporating infiltration practices, it can be quite difficult to determine which practices are best suited for a selected site. The following table highlights different infiltration practice configurations and where they are most suitable.

When not to use infiltration practices:

- Adequate pretreatment cannot be provided when high sediment loads are present
- Sites that cannot provide adequate maintenance access
- Sites that cannot provide an overflow or outlet that does not allow for the practice to drain in 72 hours
- High pollutant loads are present in the stormwater runoff (hotspots)
- Groundwater or an impermeable soil layer is within 2 feet of the bottom of the practice

Table 9-1 Infiltration practice suitability

Configuration	Suitable Sites/Applications
Infiltration Basin	Industrial sites with low sediment loads, multi-family housing
Infiltration Bed	Below other GSI practices, sites with shallow groundwater tables or impermeable soil layers, below sidewalks or plazas
Infiltration Vault	Sites requiring a large volume of water to be stored, sites with limited availability of surface space where the practice can be placed below another use
Infiltration Swale	Along roads, adjacent to parking lots, between rows of parked cars
Infiltration Trench	Under parking lots, under roads
Dry Well	Residential sites, along roads, within parking lots

9.2 Infiltration Practice Design Process

Designing infiltration practices requires careful consideration of both existing and proposed site conditions to ensure proper function of the practice. This section provides an overview of the design process and common design considerations relevant to all infiltration practices. Additionally, Table 9-2 lists chapters that are frequently cross-referenced for any applicable design methods and requirements.

Table 9-2 Cross-referenced chapters

Chapter 2	Regulatory requirements
Chapter 3	Site assessment Conceptual design
Chapter 4	Runoff volume calculations
Chapter 5	Inlet and outlet design Level spreader design Subsurface drainage system design
Chapter 6	Properties of soils and aggregates Soil water calculations Geotechnical requirements
Chapter 8	Bioretention infiltration design requirements
Chapter 10	Permeable pavement infiltration design requirements

9.2.1 Site Investigation

The ability to retain stormwater on site and to incorporate infiltration practices into the site's stormwater management approach depends on many factors that must be evaluated. A more detailed explanation on how to conduct a thorough assessment can be found in Chapter 3, Site Design and Stormwater Management. As part of the overall site assessment, the following attributes specifically apply to infiltration practices and shall be addressed:

- Zoning regulations
- Rights-of-way, road setbacks and property line setbacks
- Utility conflicts
- Buildings and other existing structures
- Surface and subsurface drainage patterns including sewer connections or other available outlets
- Open space availability and suitability
- Natural drainage characteristics and hydrologic flow paths
- Soil suitability for infiltration including the presence of contaminated soils, high groundwater tables or shallow impermeable soil layers
- Aesthetics
- Identification of stormwater hotspots or sources of sediment

Soil Evaluation

As part of the site investigation, it is imperative to evaluate the existing soils and their suitability for infiltration. Prior to beginning design, Chapter 6, Soil, Aggregates and Water should be reviewed for information on initial site assessments that can be performed to provide general information about the site's soil characteristics, as well as any geotechnical investigations that may be required.

9.2.2 Common Design Considerations

When designing infiltration practices in Detroit, there are a few common considerations that need to be considered regarding seasonal performance and environmental contamination. The section below details the most pertinent of these considerations.

Winter Conditions

Figure 9-8 Typical trash and sediment left from melting snow piles

Infiltration practices work quite well in cold climates, but require a few additional considerations to ensure their success. In general, infiltration practices are not recommended for use as snow storage areas. A frequent problem is that snow storage piles typically contain significant sediment loads which are left behind after melting and can clog infiltration practices (Figure 9-8). Ideally, runoff with heavy sediment loads should be captured by the pretreatment system prior to entering the practice. This means that snow storage should either be located upstream of the pretreatment system or downstream of the practice entirely. Another concern is that sodium based road salts have the potential to destabilize clay in subsoils through chemical reactions. The destabilization of the clay particles causes them to clog open pores or even form a cement-like surface that does not allow water to infiltrate. A third concern is that meltwater may migrate and refreeze under pavements or other infrastructure (Clark, et al., 2009).

Stormwater Hotspots

Stormwater hotspots result from activities or practices that have the *potential* to contribute significant levels of pollutants to stormwater runoff. These may or may not be regulated by State or Federal permits. Typical stormwater hotspots include:

- Vehicle maintenance, fueling, washing, and storage areas.
- Waste management areas including dumpsters.
- Outdoor storage areas for things like road salt and bulk landscape supplies.
- Heavily fertilized lawns.

Water Quality

Protection of groundwater quality is critical. When properly sited, designed, constructed, and maintained, infiltrating practices pose very little threat to nearby groundwater (SEMCOG, 2008). When potential sources of contamination are suspected or identified, there are a few additional considerations that need to be made before proceeding with design. First and foremost, is to avoid constructing infiltration practices that will directly receive stormwater runoff from areas considered 'stormwater hotspots'. Sites that include hotspots will require pretreatment systems tailored to the specific pollutants of concern, and the necessary investigation to identify these pollutants must be conducted. Hotspots are most often associated with some industrial uses and high vehicular traffic areas.

Additionally, the presence of shallow groundwater tables (less than 4 feet below the bottom of the practice) poses an additional risk by not allowing enough time for the necessary chemical, physical and biological processes to take place which filter and clean the runoff in the soil profile prior to entering the groundwater table.

Figure 9-9 Gas stations are typical stormwater hotspots

Contaminated Sites

A site is contaminated when one or more contaminants in the soil, sediment or water exceeds a concentration that may pose a threat to human health and the environment. More information on hazardous substances regulated by the State of Michigan under Part 201 Generic Cleanup Criteria and Screening Levels can be found here: http://www.michigan.gov/deq/0,4561,7-135-3311_4109-251790--,00.html

Examples of sites that may be contaminated include: Superfund sites; brownfield sites; treatment, storage and disposal facilities regulated under the Resource Conservation and Recovery Act (RCRA); and underground storage tank (UST) sites.

Infiltrating water in locations that may mobilize contaminants is not allowed. However, the presence or suspicion of contaminated soils on a site does not necessarily preclude the use of infiltrating practices. Special design considerations that ensure infiltrating water will not mobilize contaminants can be used following additional geotechnical studies and analyses to confirm the safety of the proposed practices. This is often accomplished by locating the infiltrating practices down gradient from the contaminated soil. Examples of additional resources on infiltrating stormwater practices on contaminated sites include:

- *Implementing Stormwater Infiltration Practices at Vacant Parcels and Brownfield Sites* (USEPA, 2013).
- *Design Principles for Stormwater Management on Compacted, Contaminated Soils in Dense Urban Areas* (USEPA, 2008).
- *Case Studies for Stormwater Management on Compacted, Contaminated Soils in Dense Urban Areas* (USEPA, 2008).

Underground Injection Control Permits

Most green stormwater infrastructure practices are not regulated under the Underground Injection Control (UIC) program. In a memorandum dated June 13, 2008, EPA described the situations where the UIC program applies to various stormwater

Class V Wells

A well-used for the management of stormwater is generally considered Class V if its design directs stormwater to any bored, drilled, driven shaft, or dug hole that is deeper than its widest surface dimension, or if it has a subsurface fluid distribution system. Class V wells are subject to the Underground Injection Control program.

infiltration practices (Boormazian & Heare, 2008). Practices such as rain gardens, bioretention areas, vegetated swales, stormwater wetlands, and permeable pavement are typically not regulated under the UIC program. Systems that are deeper than they are wide or that include a subsurface distribution system are subject to the UIC program. Drywells, seepage pits, improved sinkholes, and commercially manufactured stormwater infiltration devices are all generally considered Class V wells since their designs often meet the Class V definition. The UIC program is administered by USEPA Region V and Michigan Department of Environmental Quality (https://www.epa.gov/uic).

9.2.3 Common Design Elements

Two major design elements that significantly affect the performance of infiltration practices concern the management of sediments and even distribution of incoming runoff. Ensuring that water entering the practice is evenly spread out across the entire surface area of the practice is necessary for maximum infiltration and the prevention of sediment concentration in isolated areas. The following section recommends methods to help maximize the performance of infiltration practices.

Level Spreaders and Berms

Because infiltration practices can vary significantly in area and in flow length, water entering the practice has the potential to transition from sheet flow to shallow concentrated flow. When this happens, interventions must be designed to prevent the water from naturally concentrating into channels to improve infiltration.

Figure 9-10 Typical level spreader

Level spreaders are used to spread out water over large (often vegetated) surfaces to reduce stormwater velocities and more evenly distribute water to a practice. Information on sheet flow, shallow concentrated flow and the design of level spreaders are discussed in Chapter 5, Drainage Conveyance.

Infiltration Berms are linear built-up earthen embankments with sloping sides placed along (i.e. parallel to) existing site contours, in a moderately sloping area (SEMCOG, 2008). The berms are intended to retain, slow down, or divert stormwater as it flows down the slope. The berms in effect act as speed bumps for the water; slowing it down and giving the water a chance to infiltrate into the soil.

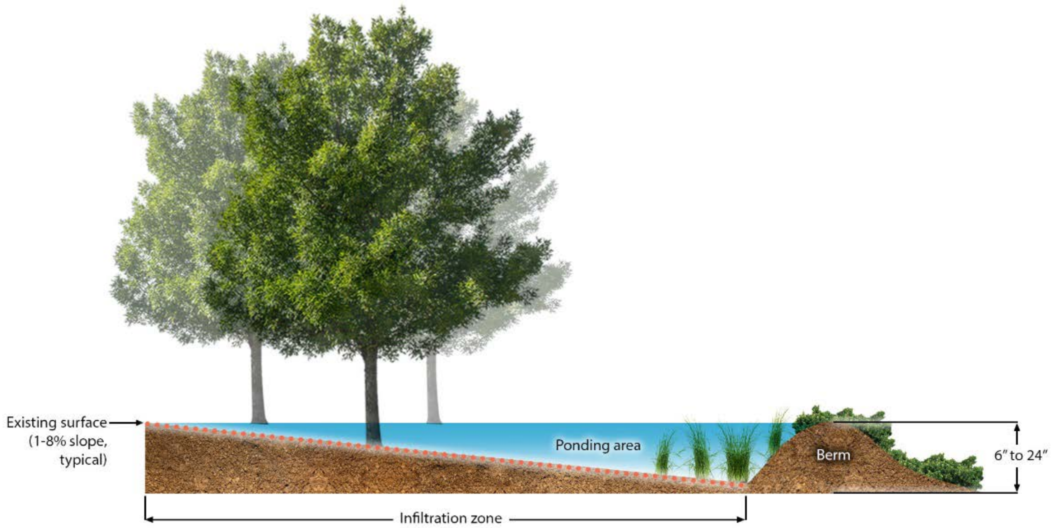

Figure 9-11 Section of an infiltration berm

Sediment Pretreatment

A major cause of failure for infiltration practices is clogging of the infiltration area due to sediment in the stormwater runoff. To minimize the likelihood of clogging, pretreatment should be utilized for most infiltration practices. Pretreatment devices that operate effectively in conjunction with infiltration include grass swales, vegetated filter strips, settling chambers, oil/grit separators, sediment sumps, and manufactured treatment systems. Selection of pretreatment should be guided by the *pollutants* of greatest concern, site by site, depending upon the nature and extent of the land development under consideration. Selection of pretreatment techniques will vary depending upon whether the pollutants are of a particulate (sediment, phosphorus, metals, etc.) versus soluble (nitrogen and others) nature. Types of pretreatment (i.e., filters) should be matched with the nature of the pollutants expected to be generated.

Figure 9-13 Installation of typical manufactured treatment device

Designing a forebay for an infiltration practice is the same as a forebay for a large detention practice, refer to Chapter 7, Detention Practices. Use of a manufactured treatment device is discussed in Chapter 14, Manufactured Treatment Systems. A sump pretreatment approach may be suitable for small practices; sumps are discussed in Chapter 5, Drainage Conveyance.

Figure 9-12 Sediment forebay

9.3 Layout

The layout of infiltration practices needs to account for a variety of factors during the design process to ensure that sufficient space is available and the practice is accessible for routine maintenance activities. The section below details the most common layout considerations.

9.3.1 Location

The following criteria shall be used to define the layout and placement of all infiltration practices.

- Infiltration practices shall not be constructed on steep slopes (greater than 20%), nor should slopes be significantly altered or modified to reduce the steepness of the existing slope for the installation of an infiltration practice.
- The removal of mature trees for the explicit purpose of installing infiltration practices is strongly discouraged. Mature trees play a significant role in the natural hydrologic cycle and should be protected whenever possible. See Chapter 3, Site Design and Stormwater Management for additional recommendations on how to preserve and protect existing vegetation.
- Infiltration practices shall not be located within a defined 100-year floodplain.
- Infiltration practices shall be located such that infiltrating water will not mobilize pollutants from contaminated areas.
- Infiltration practices shall have sufficient easements for maintenance purposes. Easements shall be sized and located to accommodate access and operation of equipment, spoils, deposition, and other activities identified in the development's stormwater management plan.

Setback Requirements

Because infiltration practices increase the amount of water present in the soil profile, it is important to maintain minimum separation distances between these practices and specific site features. Saturated soils can negatively affect building foundations, water supply wells, and nearby off-site features. Refer to Table 9-3 for required infiltration practice setbacks.

Table 9-3 Setback Distances

Setback from	Minimum horizontal distance (feet)
Property line	10
Building foundation[1]	10
Municipal sanitary or combined sewer[2]	10
Public water supply well[3]	50

1. Minimum with slopes directed away from building
2. The Department may stipulate additional setback requirements from certain municipal sewers
3. At least 200 feet from Type I or IIa wells, 75 feet from Type IIb and III wells (MDEQ Safe Drinking Water Act, PA 399)

Maintenance Access

Regular maintenance is a key component to ensuring the long-term functionality of infiltration practices. Typical maintenance activities such as sediment removal, trash removal and inspection of structures, requires considerations for access during the design. A full list of maintenance activities is presented in Section 9.6. It is recommended that this section be reviewed to ensure that proper access is provided for their execution.

9.3.2 Sizing the Practice

Once a suitable location has been determined and the appropriate configuration has been selected, the area of the practice footprint can be determined. Determining the area is an iterative process that frequently requires further modifications to many design elements before a suitable design solution can be found. The following sections and chapters within the manual should be referenced for additional sizing criteria:

- Section 9.4 below includes design standards and requirements for each infiltration practice configuration.
- Infiltration calculation methods are provided in Chapter 6, Soil, Water and Aggregates.
- Detention sizing and detention outlet structure calculations are provided in Chapter 7, Large Detention Practices.
- Chapter 8, Bioretention details a step by step approach for sizing an infiltration practice with multiple layers which may be helpful for some of the infiltration practice configurations.

9.4 Design Standards

Design standards and requirements are provided in this section. General requirements are presented first, which apply to all infiltration practices. Following the general requirement section, standards that are unique to each type of practice are presented.

The design and construction of infiltration practices must meet all the general requirements and the practice specific requirements.

Flow Configurations

Offline System (Passive System)

Stormwater flows into the practice until capacity is reached. Excess volume either flows out the downstream end toward the surface storm drainage system or bypasses the practice. Excess water shall not back up into a street.

Online System

All stormwater runoff is directed into the practice. An overflow system manages additional flows above the maximum ponding depth. The overflow device conveys higher flows to the collection system and must be protected from erosion.

Infiltration practices are recommended to be designed as offline systems, i.e. low flows are diverted into the infiltration practice area whereas high flows bypass the infiltration practice altogether. In addition, a way to dewater the system is recommended to be included in the design in case of clogging. This will allow for easier maintenance in case of problems.

9.4.1 General Requirements

This section presents general requirements that must be met for all types of infiltration practices.

- The lowest elevation within an infiltrating practice shall be at least 4 ft. above the seasonal high-water table or any other impermeable soil layer. A 2-ft. minimum clearance may be used with approval from the Department. The performance of the infiltrating practice may be reduced if a minimum clearance to the seasonal high-water table or infiltration limiting layer is less than 4 ft.
- The bottom of the infiltrating practice shall have a maximum 1 percent slope. The preferred bottom slope is flat (a slope of zero percent).
- Provide appropriate pretreatment prior to infiltration. The type of pretreatment should be matched with the nature of the pollutants expected to be generated.
- Infiltrating practices shall be designed to completely dewater within 72 hours of the end of the storm event. Water on the surface should generally be drained within 24 hours unless designed as an extended detention system.
- A minimum one (1) foot of freeboard is required above the 100-year flood elevation of the infiltration practice and the low entry elevation of structures near the practice. If building foundation drains are gravity discharged to the infiltration practice, then a minimum of one (1) foot of freeboard is required above the 100-year flood elevation of the infiltration practice and the basement floor elevation of the nearby buildings.
- Permanent access for maintenance is required. Where forebays, sumps, or manufactured treatment devices are used for pretreatment, vehicular access shall be provided at least nine feet wide, have a maximum slope of 15 percent,

and be stabilized for vehicles. All structures associated with the practice including any inlets, outlets, overflows, subsurface drainage system cleanouts, and observation wells shall be accessible for inspection.

9.4.2 Basins

The following section details additional requirements that are specific to the design of infiltration basins. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to infiltration basins contained within this chapter and manual.

- The design shall prevent erosion throughout the entire basin including but not limited to the inlet(s), forebay, outlet works, emergency overflow, basins sides and embankments. Erosion may be controlled with hard armoring techniques or vegetation.
- Practices designed to be online shall be designed to safely pass a 100-year storm. If the 100-year storm is not specified as part of the site-specific design requirements an emergency outlet or spillway capable of conveying the 100-year design storm shall be included in the design.
- Side slopes shall not be steeper than 3H:1V. Terraced side slopes (Figure 9-14) are allowed however the maximum vertical rise is limited to 18 inches.

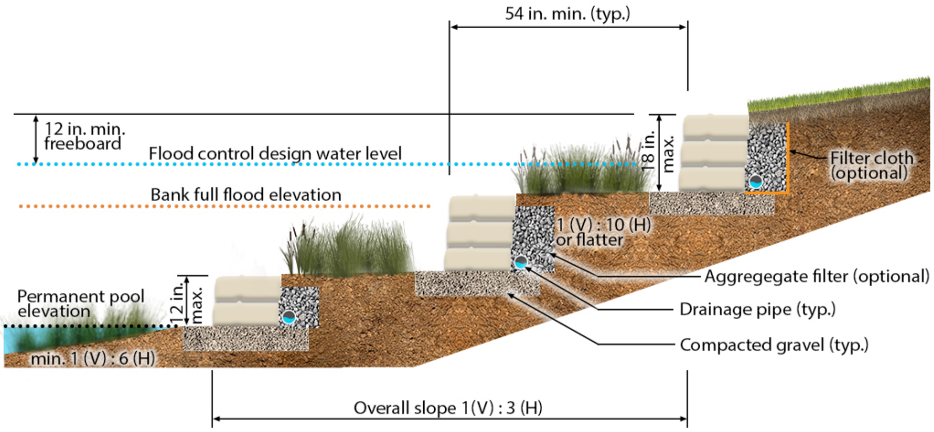

Figure 9-14 Terraced side slopes on an infiltration basin

- Infiltration basins intended to also provide detention storage shall be designed to meet the requirements of Large Detention Practices (Chapter 7).
- A landscaping plan is required for infiltration basins. All infiltration practices that include vegetation shall follow Chapter 8, Bioretention for the requirements and recommendations related to vegetation and associated construction practices. The landscape plan shall include at a minimum the following:
    - Existing site conditions and vegetation (e.g. trees 6-inch caliper and larger) that may be affected by the project;
    - Plan view of the infiltration basin, including one-foot grading contours;

    - Elevations in the infiltration basin, including the bottom elevation and all the maximum water surface elevations based on the hydrologic requirements;
    - Identification of planting zones based on levels of inundation;
    - Vegetation selection, plant spacing and applicable depths;
- Woody vegetation may not be planted on nor allowed to grow within 15 feet of the toe of an embankment (USACE, 2014).
- Woody vegetation may not be planted on nor allowed to grow within 25 feet of the emergency overflow.

9.4.3 Beds

The following section details additional requirements that are specific to the design of infiltration beds. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to infiltration beds contained within this chapter and manual.

- Infiltration beds placed in areas intended for vehicular traffic shall be designed and constructed for a minimum HS-20 loading.
- A minimum of one (1) observation well shall be provided consisting of an anchored, vertical perforated PVC pipe fitted with a lockable cap installed flush with or above the ground surface. The observation well shall extend to the bottom of the infiltration bed.
- When perforated pipes are installed in the infiltration bed to distribute flow evenly over the entire bed bottom, the following are required:
    - Perforated pipes shall be laid flat to uniformly distribute the water.
    - Cleanouts and/or inlets shall be provided to access the piping system.
    - The maximum bend angle allowed on the piping is 45 degrees.
    - Distribution piping that connects to a downstream collection sewer shall be solid wall pipe with watertight joints for a minimum of 10-ft from the point of connection and include an anti-seepage collar. When connecting to the municipal combined sewer system, the design shall include a backflow preventer and an odor trap.

9.4.4 Vaults

The following section details additional requirements that are specific to the design of infiltration vaults. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to infiltration vaults contained within this chapter and manual.

- A pretreatment system is required with all subsurface infiltration vault practices.
- All subsurface infiltration vault practices shall have a means to inspect and maintain the entire system.
- Subsurface infiltration vault practices shall be designed and constructed for a minimum HS-20 loading.

- All manufacturer recommendations shall be adhered to for subsurface infiltration vault practices.
- When constructing the outlet from an infiltration vault into a downstream collection sewer, the piping shall be solid wall pipe with watertight joints for a minimum of 10-ft from the point of connection with the sewer and include an anti-seepage collar. All connections to a municipal combined sewer system shall include a backflow preventer and an odor trap.

9.4.5 Swales

The following section details additional requirements that are specific to the design of infiltration swales. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to infiltration swales contained within this chapter and manual.

- Side slopes shall not be steeper than 3H:1V. Terraced side slopes are allowed however the maximum vertical rise is limited to 18 inches.
- The design shall prevent erosion throughout the swale including, but not limited to, the inlet(s), forebay, outlet works, emergency overflow, swale sides, and embankments. Erosion may be controlled with hard armoring techniques or vegetation.
- A landscaping plan is required for infiltration swales. The landscape plan shall include at a minimum the following:
    - Existing site conditions and vegetation (e.g. trees 6-inch caliper and larger) that may be affected by the project;
    - Plan view of the infiltration swale, including one-foot grading contours;
    - Elevations in the infiltration basin, including the bottom elevation and all the maximum water surface elevations based on the hydrologic requirements;
    - Identification of planting zones based on levels of inundation;
    - Vegetation selection, plant spacing and applicable depths.

9.4.6 Trenches

The following section details additional requirements that are specific to the design of infiltration trenches. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to infiltration trenches contained within this chapter and manual.

- A minimum of one (1) observation well shall be provided consisting of an anchored, vertical perforated PVC pipe fitted with a lockable cap installed flush with or above the ground surface. The observation well shall extend to the bottom of the infiltration bed.
- When perforated-pipes are installed in the infiltration trench to distribute flow evenly over the entire bed bottom, the following are required:
    - Perforated pipes shall be laid flat to uniformly distribute the water.
    - Cleanouts and/or inlets shall be provided to access the piping system.

  - The maximum bend angle allowed on the piping is 45 degrees.
  - Piping systems connecting to downstream collection sewers shall be solid wall pipes with watertight joints for a minimum of 10-ft and include an anti-seepage collar. When connecting to a combined sewer system a backflow preventer and odor trap shall be provided.

9.4.7 Dry Wells

The following section details additional requirements that are specific to the design of dry wells. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to dry wells contained within this chapter and manual.

- Dry wells shall be designed for inspection access. Manufactured systems including a removable lid may suffice for inspection access or an observation well may be used.
- The design depth of a dry well should consider frost depth to prevent frost heave.
- When an observation well is provided, it shall consist of an anchored, vertical perforated PVC pipe fitted with a lockable cap installed flush with or above the ground surface. The observation well shall extend to the bottom of the dry well.
- Dry wells within areas intended for vehicular traffic shall be designed and constructed for a minimum HS-20 loading.

9.5 Construction Considerations

Before construction begins, the entire area draining to the infiltration practice should be stabilized. If possible, a berm should be placed around the infiltration practice during construction to prevent sediment accumulation on the infiltration surface. During excavation of the infiltration practice, place excavated material in an area where it will not be washed back into the practice footprint if a storm occurs.

Construction considerations for soil compaction of the area intended for infiltration are discussed in other chapters. All the same construction considerations apply regardless if the infiltrating stormwater practice is bioretention, permeable pavement or an infiltration practice discussed in this chapter.

- Chapter 6. Soil, Aggregates and Water – discusses general information regarding compaction of soil and the impact on the soil structure and hydraulic conductivity.
- Chapter 8. Bioretention – discusses general strategies to minimizing compaction of practices with small to medium size footprints.
- Chapter 10. Permeable Pavements – addresses ways of reducing compaction on large infiltration practices more typical of permeable pavement installations.

9.6 Operation and Maintenance

Infiltration practices require regular maintenance to ensure proper function. Table 9-4 lists specific tasks to be completed for infiltration practices.

To prevent clogging and damage to vegetation, salt, sand and other deicers should not enter an infiltration practice. Snow disposal sites should generally not be in an area that drains directly to an infiltration practice.

Table 9-4 Inspection and Maintenance Tasks for Infiltration Practices

Task	Frequency	Indicator maintenance is needed	Maintenance notes
Catchment inspection	Weekly or biweekly with routine property maintenance	Excessive sediment, trash, or debris accumulation on the surface of infiltration practice	Permanently stabilize any exposed soil and remove any accumulated sediment. Adjacent pervious areas might need to be regraded.
Inspect dewatering duration	After events over 1 inch	If practice does not drain within 72 hours	Monitor drawdown time after significant rain events.
Inspect pretreatment/ forebay	2-4 times/year	Internal erosion or excessive sediment, trash, or debris accumulation	Check for sediment accumulation to ensure that flow into the infiltration practice is as designed. Remove any accumulated sediment.
Remove sediment from surface of infiltration practice	As needed	Extended drawdown times; excessive sediment	Scrape bottom, remove and properly dispose of sediment; restore original cross section
Remove accumulated trash and debris	Quarterly	Accumulation of trash and debris in infiltration area	Trash and debris should be removed to reduce risk of clogging. Properly dispose of all trash and debris removed from site.

9.7 Design Checklist

The design of infiltration practices typically involves a few iterations, design of several individual components, and frequent modifications during the design process. To ensure that the practice has been properly designed, the following checklist shall be used upon completion of the design, but before construction drawings have been finalized. This checklist can then be included as part of the Post-Construction Stormwater Management Plan along with any required calculations to document the design.

Insert checklist here.

9.8 References

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10. Permeable Pavement

Permeable pavement is a versatile GSI practice that can be incorporated into a variety of situations where both stormwater management and transportation infrastructure are required. Permeable pavements work by infiltrating stormwater runoff through the pavement surface and then temporarily storing it for infiltration into the subgrade or controlled release back into a stormwater conveyance system. Because there are a variety of options available for the type of permeable pavement surface material, these practices can be located on nearly any site, regardless of size or use.

Contents

10.1 Introduction

Permeable pavement is a highly versatile stormwater management practice that allows streets, parking lots, sidewalks, and other impervious covers to retain the infiltration capacity of underlying soils while maintaining the structural and functional integrity of traditional pavements. Permeable pavements have small voids or aggregate-filled joints that allow water to drain through the surface and into an aggregate reservoir. Stormwater collected in the aggregate storage layer can then infiltrate into underlying soils or drain at a controlled rate via underdrains to other downstream stormwater control systems. Permeable pavement practices can be designed to operate as underground detention if the native soils do not have sufficient infiltration capacity, if there are contaminated soil conditions, or if infiltration would negatively impact adjacent structures. In many cases, especially where space is limited, permeable pavement may be a cost-effective solution relative to other practices because it doubles as both transportation infrastructure and stormwater infrastructure.

Advantages

- Can be used in place of traditional pavement surfaces
- Can fit into spaces of almost any size and integrated into many different site layouts
- Reduces ponding and icing associated with traditional hardscape surfaces
- Provides better conditions for trees, reduced heat island effect, quieter vehicular traffic, and reduced vehicular glare compared to traditional standard asphalt

Limitations

- Requires special consideration if used in high traffic or heavy industrial areas where heavy sediment or gross pollutant loads may be present
- Requires frequent maintenance with specialized equipment to maintain performance
- Porous asphalt or pervious concrete may degrade more rapidly if located in areas with frequent vehicular turning

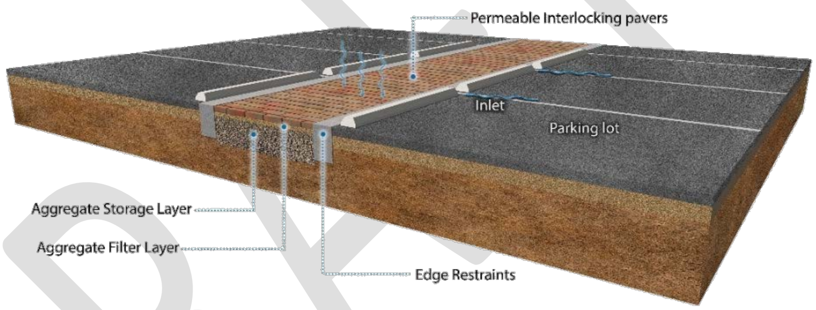

Figure 10-1 Permeable interlocking concrete pavers in a parking lot

10.1.1 Major Components

The major components of a permeable pavement system are:

Permeable Pavement Surface Layer – a structural surface that allows the water to flow through while still providing the function of traditional impervious pavements.

Aggregate Filter Layer – a layer of open graded aggregate to prevent migration of fines moving into or through adjacent material layers. They are commonly used below the permeable pavement surface layer to provide a platform for the porous surface material, to prevent fines from a paver bedding layer from migrating into the aggregate storage layer below, and to separate the subgrade from the bottom of the aggregate storage layer.

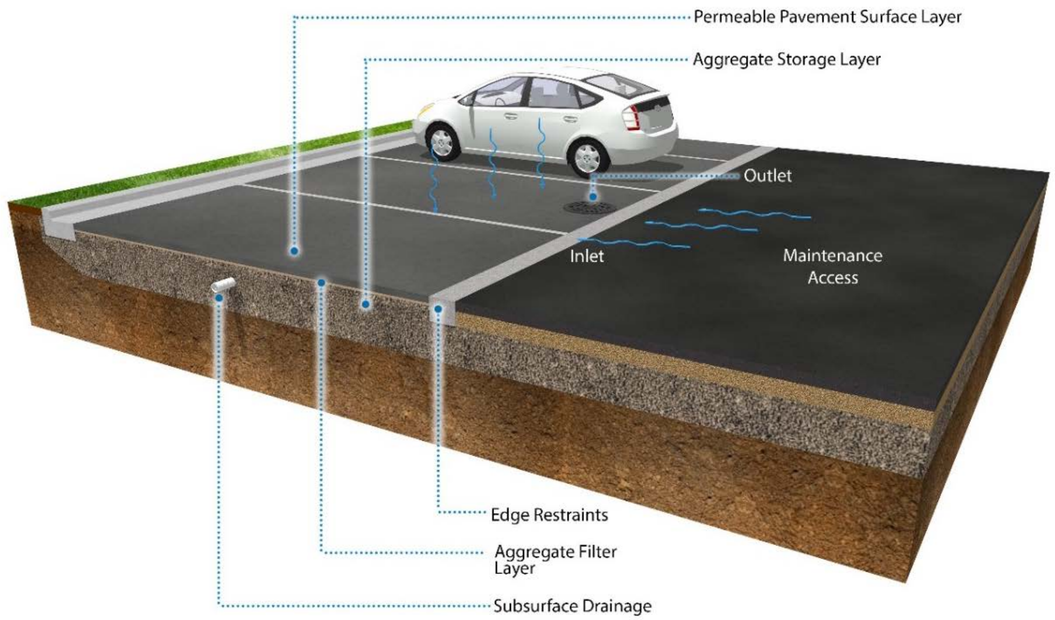

Figure 10-2 Components of a typical permeable pavement practice

Aggregate Storage Layer – a layer of open graded aggregate added below the aggregate filter layer to provide stormwater storage volume. In flexible pavement systems, this layer is also required to distribute loads to the subgrade.

Subsurface Drainage – permeable pavement practices commonly include subsurface drainage systems in situations where native soils have low permeability, when an impermeable liner is required, or to control subsurface infiltration.

Liner – a special layer placed on the sides, the bottom, or both. Liners may be impermeable or permeable and are incorporated based on the type of practice being designed and site conditions present.

Inlet and Pretreatment – an inlet is a structure designed to direct the stormwater runoff into the stormwater practice. Pretreatment systems capture pollutants from stormwater runoff before delivery to the primary storage or infiltration area. Due to the nature of porous pavements, inlets are typically sheet flow and pretreatment systems are very uncommon.

Outlets and Overflows –hydraulic structures designed to control the release of water from the practice and to act as emergency overflows.

Edge Restraints – suitable edge restraints are critical to the satisfactory performance of certain permeable pavement systems. Installations of pavers and permeable asphalt require the use of edge restraints. Pavers must abut tightly against the restraints to prevent rotation, settlement or spreading of joints. It is also recommended that edge restraints be used in pervious concrete installations where edge cracking may be a concern. Edge restraints can often be designed to function as the inlet for the practice.

Maintenance Access – a dedicated access to the practice to allow for maintenance activities. Access is required for periodic maintenance of the surface layer and to service cleanouts for the subsurface drainage system, inlet, pretreatment, and outlet when present.

10.1.2 Surface Options

For most pavement applications, a permeable surface alternative is available in the same material as its traditional impervious counterpart. For streets and parking lots, permeable pavers, pervious concrete and porous asphalt all work well but have their own design limitations and maintenance considerations. The choice often comes down to cost and appearance. The developer, designer and future site owner/manager should thoroughly investigate each pavement option by visiting installations in the area and talking to those who have designed, constructed and maintained them. Brief descriptions of common permeable pavement surface types are described below:

Porous Asphalt

Porous asphalt pavement consists of fine- and course-aggregate stone bound by a bituminous-based binder. The amount of fine aggregate is reduced to allow for a larger void space of typically 15 to 20 percent. The thickness of the asphalt depends on the traffic load but usually ranges from 3 to 7 inches. A required underlying aggregate base course is used to add strength and doubles as the aggregate storage layer.

Figure 10-3 Example of Porous Asphalt

Refer to the National Asphalt Pavement Association (http://www.asphaltpavement.org/) for additional information, including a design guide on porous asphalt pavements (NAPA, 2008).

Pervious Concrete

Pervious concrete is a mixture of Portland cement, fly ash, washed gravel, and water. The pavement typically has 15% to 25% interconnected void space that allows rapid infiltration of stormwater to the aggregate storage layer and underlying soil (Eisenberg, Lindlow, & Smith, 2015). A fine, washed gravel, less than ½ inch in size (No. 8 or 89 stone), is added to the concrete mixture to increase the void space (GCPA, 2006). Admixtures are then used to improve bonding and strength of the pavement. Pervious concrete pavements are typically laid with a 4 to 8-inch thickness on top of an aggregate storage layer to increase water storage and infiltration. The compressive strength of pervious concrete typically ranges from 400 to 4,000 pounds per square inch (psi) (NRMCA, 2004).

Figure 10-4 Example of Pervious Concrete

Refer to the Michigan Concrete Association for additional information on pervious concrete (https://www.miconcrete.org/pervious). Also refer to the National Ready Mixed Concrete Association (NRMCA) for additional information on pervious concrete, including a certification program for contractors (https://www.nrmca.org/Education/Certifications/Pervious_Contractor.htm).

Permeable Interlocking Pavers

Permeable interlocking pavements are available in many different shapes and sizes. When installed, the blocks form patterns that create openings through which rainfall and snowmelt can infiltrate. The openings, which are generally 8 to 20 percent of the surface area, are either filled with a small drainage aggregate or are held open by special spacers on the pavers. ASTM C936 specifications state that the pavers shall be at least 2.36 inches thick with a compressive strength of 8,000 psi or greater.

Figure 10-5 Example of Permeable Interlocking Concrete Pavers

Refer to the Interlocking Concrete Pavement Institute (ICPI) (https://www.icpi.org/paving-systems/permeable-pavers). and the Brick Industry Association (http://www.gobrick.com/) for additional information on permeable interlocking concrete and clay pavers.

Grid Pavement Systems

Grid pavement systems, also called geocells or turf pavers, consist of interlocking units typically constructed from concrete or a flexible plastic that allow for infiltration through large gaps filled with gravel or topsoil planted with turfgrass. The empty grids are typically 90 to 98 percent open space, so void space depends on the fill media (Ferguson, 2005). To date, no uniform standards exist; however, one product specification defines the typical load-bearing capacity of empty flexible plastic grids at approximately 2,000 psi. That value increases up to 5,500 psi when filled with various materials (Invisible Structures, Inc., 2015).

Figure 10-6 Example of Grid Pavement Systems

Reinforced turf pavement is an excellent choice for emergency vehicle (e.g., fire truck) access lanes or overflow parking areas because these areas provide excellent retention capabilities while providing green space, and often require less aggregate and other structural inputs, which lowers costs.

Figure 10-7 Example of Open Graded Aggregate as a Pavement Surface

Permeable Aggregate

Use of a washed, open graded aggregate as a permeable pavement surface is not a widespread practice. However, this surface option tends to provide the highest permeability and lowest cost of any of the other surface options (Ferguson, 2005). Permeable aggregate is not ADA compliant, and therefore, works best in parking lots and driveways. Typically, 8 to 10 inches of an open graded drainage course is placed and compacted to achieve a smooth driving surface.

Pervious Pavers

Figure 10-8 Example of Pervious Paver

Pervious pavers allow water to percolate through the surface of the pavers, as opposed to through the joints in between the pavers. Typically constructed from natural stone that is bonded together with a polymer or cement, the pavers themselves have around 20% to 40% void space with flow-through rates as high as 5,000 inches per hour when initially installed (Eisenberg, Lindlow, & Smith, 2015). While their performance is like that of pervious concrete, an additional level of quality control exists during the manufacturing process which can avoid failures that may occur with improper installation of pervious concrete. Furthermore, polymer bound pavers utilize binders that are typically immune to the effects of freeze/thaw, deicing salts, and show some resistance to petroleum products (Eisenberg, Lindlow, & Smith, 2015).

Pervious pavers work well in high pedestrian traffic areas due to their lack of joints. They can also be used in high frequency traffic applications where low speeds are anticipated. There are currently no ASTM standards for pervious pavers.

Rubber Overlay and Pavers

Figure 10-9 Example Rubber Overlay

Rubber can be used both as a pervious overlay pavement and as composite permeable pavers. The pervious overlays are typically constructed from recycled rubber granules, aggregate and a proprietary binder. This mix is then poured in place to a desired thickness over a permeable base. Rubber overlay pavement is appropriate for light traffic and low speed applications such as pathway, courtyards, driveways, sidewalks and playgrounds.

Figure 10-10 Example Rubber Composite Permeable Paver

Source: AZEK© Pavers

Rubber pavers are made from up to 95% recycled materials and function similarly to permeable interlocking concrete pavers with open joints for infiltration. They are typically used in pedestrian or light automobile traffic applications, but come with the advantage of being one-third the weight of standard concrete pavers (Eisenberg, Lindlow, & Smith, 2015).

10.1.3 Landscape Context

Many times, determining how permeable pavement will be included in the site design is a critical first step. Site and soil evaluation will help determine retention potential, influence location and sizing of the infiltration bed, and pavement drainage options. Areas with fill (especially compacted fill) may have low infiltration potential and should be avoided. Expected vehicle types and traffic patterns will help evaluate location and choice of permeable pavement surface within the site's overall pavement footprint.

Figure 10-11 Permeable Parking Lane

Within Roadways

Permeable pavement can be incorporated in the parking lane of a traditional asphalt road. These areas can be designed to provide treatment for both the parking lane and traditional roadway runoff.

Figure 10-12 Permeable Parking Lot

Within Parking Lots

Parking lots are a popular application for permeable pavements. The permeable pavement may cover the entire parking lot surface or the lot may have a combination of permeable and standard pavements; oftentimes the driving lanes are paved with standard paving that drains to permeable parking stalls.

Figure 10-13 Permeable Sidewalk

Sidewalks and Pedestrian Plazas

Permeable pavement can be effective for pedestrian uses - most types of permeable surface courses are ADA compliant. Sidewalks can be constructed of pervious pavement materials to reduce runoff in highly impervious areas. This can be effective in malls, plazas, courtyards, and other outdoor hardscapes with low sediment loads. Care should be taken during site layout to accommodate maintenance.

Figure 10-14 Permeable Access Road

As Access Roads

Permeable pavement can also be used in areas that receive little traffic, such as fire lanes or vegetated shoulders for temporary parking.

Figure 10-15 Permeable Playground

When not to use permeable pavement:

- Adequate pretreatment cannot be provided when high sediment loads are present.
- Sites with no maintenance access.
- Sites that cannot provide an overflow or outlet that does not allow for the practice to drain in 72 hours.
- Infiltration practices should not be used when contaminated soils are present, where groundwater contamination is likely due to spills or hotspots (truck stops, fueling station, etc.), or within design setback distances.
- Sites with overhanging trees that deposit detritus in amounts beyond what maintenance can handle.

Recreational Facilities

Porous asphalt can be used for recreational facilities such as basketball courts, which neighboring residents report results in lower sound attenuation as compared to traditional asphalt basketball courts.

10.1.4 Site Suitability

Permeable pavement is an excellent alternative to impermeable paving and helps to meet retention (volume reduction) requirements in many projects that are adding or renovating parking, driveways, streets or alleys. However, permeable pavement is not always an appropriate choice in every application. Table 10-1 lists some of the current constraints and limitations of the different permeable pavement surfaces that may preclude their use. Many of these pavements are actively being researched and improved, so it is important to check with the latest industry standards for appropriate applications.

Permeable pavement is typically designed to treat stormwater that falls on the actual pavement surface area and has been used at commercial, institutional, and residential sites in spaces that are traditionally impervious. Runoff from pervious surfaces like lawns or high-sediment areas should be prevented, and permeable pavement should not be installed in areas prone to flooding with sediment-laden water (e.g., floodplains) because excessive sediment can prematurely clog the pores. Overhanging trees should also be avoided to reduce the deposition of detritus on the pavement surface, which can be ground into joints and pores if not routinely removed.

Figure 10-16 Permeable pavement with significant leaf litter

Table 10-1 Permeable pavement suitability

Surface Option	Suitable Sites	Constraints & Limitations
Porous asphalt	Overflow parking, primary parking, sidewalks & pathways, drives and aisles, low volume roads with low design loads	Heavy duty porous asphalt has limited availability, additional snow plowing considerations are necessary to prevent damaging the surface, not recommended in areas with repetitive turning actions, not recommended for installation during periods of low temperatures (<55 degrees F)
Pervious concrete	Overflow parking, primary parking, sidewalks & pathways, drives & aisles, access drives	Not yet recommended for highway use, tends to ravel & deteriorate under high turning loads, deicing materials can damage the cement, not recommended for installation during periods of low temperatures (<40 degrees F) or high temperatures (>90 degrees F), typical cure time of 7 days
Permeable interlocking concrete pavers	Overflow parking, primary parking, sidewalks & pathways, drives & aisles, access drives, roads, loading areas, frequent truck traffic, industrial sites with non-hazardous materials, low volume roads with low speeds	Some manufactured products may require subgrade compaction requirements that could reduce infiltration rates of native soils
Grid pavement systems	Overflow parking, sidewalks & pathways, drives & aisles, access drives	Not recommended for high volume traffic, very sensitive to sediment loads, no deicing salts when grass pavements are used, grass pavements cannot be used until grass is established, snow can only be plowed if blade is set above surface, rotary brushes should not be used for snow removal
Permeable aggregate	Overflow parking, primary parking, drives & aisles, access drives	Not ADA compliant
Pervious pavers	Overflow parking, primary parking, sidewalks & pathways, drives & aisles, access drives	Concrete pavers can be damaged by frequent applications of deicing salts
Rubber overlay and pavers	Overflow parking, primary parking, sidewalks & pathways, drives & aisles	Not recommended for high volume traffic or speeds over 25 mph

10.2 Permeable Pavement Design Process

Permeable pavement practices require careful consideration of both existing and proposed site conditions to ensure proper function of the practice. This section provides an overview of the design process with detailed discussions of the hydrologic design and material selection for each component in the subsequent sections. Additionally, Table 10-2 lists chapters within the Manual that are frequently cross-referenced for any applicable design methods and requirements.

Table 10-2 Cross-referenced chapters

Chapter 2	Regulatory requirements
Chapter 3	Site assessment Conceptual design
Chapter 4	Runoff volume calculations
Chapter 5	Inlet and outlet design Subsurface drainage system design
Chapter 6	Properties of soils and aggregates Soil water calculations Geotechnical requirements

10.2.1 Site Investigation

The ability to retain stormwater on site and to incorporate permeable paving practices into the site's stormwater management approach depends on several factors that must be evaluated for each development site. A more detailed explanation on how to conduct a thorough assessment can be found in Chapter 3, Site Design and Stormwater Management. As a part of the overall site assessment, the following attributes specifically apply to permeable paving practices and should be addressed at a minimum:

- Rights-of-way, road setbacks and property line setbacks
- Buildings and other existing structures
- Underground structures or utilities
- Surface and subsurface drainage patterns including existing and proposed slopes, sewer connections or other available outlets
- Presence of excessive sediment or debris that may clog the pavement surface, especially an abundance of mature trees
- Soil suitability for infiltration including contaminated soils, high water tables or shallow bedrock
- Soil suitability for desired structural loading needs of the design
- Aesthetics

10.2.2 Geotechnical Evaluation

Once permeable pavement has been selected, it is required that the in-situ soils be tested before the practice can be designed. Performing soil tests during the conceptual and preliminary design phases will ensure that the proposed permeable pavement practice is optimized to actual site conditions and to prevent costly change orders resulting from poorly estimated soil parameters. Chapter 6, Soil, Aggregates and Water should be reviewed prior to beginning design for information on initial site assessments that can be performed to provide general information about the site's soil characteristics, as well as any geotechnical investigation that are required for infiltration calculations and structural design.

10.3 Permeable Pavement Structure

All pavements, regardless if they are permeable or traditional dense pavements, must bear the traffic loads imposed on them. The pavements must function year-round given Detroit's climate and soil conditions. The traffic load imposed depends on the intended use, e.g. pedestrian, bicycle, passenger vehicles, or heavy trucks.

Pavement design is commonly based on the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO, 1993) or the 2015 AASHTO Mechanistic-Empirical Pavement Design Guide (AASHTO, 2015). The design process accounts for the expected lifespan of the pavement, the reliability, the design loads, and the subgrade. The subgrade is the soil underneath the pavement structure which ultimately bears the weight of the pavement and the traffic loads.

The subgrade is often the weakest structural component. The pavement system is designed to spread the load over the subgrade to the extent the soil can bear the load without deforming. In Detroit, the pavement is also designed to counteract the soil's tendency to heave due to winter temperatures.

The stability and strength of a soil is fundamentally dependent on the type of soil, the moisture content and the level of compaction. One of the most effective ways of increasing a soil's strength and stability is to compact it and reduce the moisture content. Drying out and compacting the subgrade is a standard approach for the design of traditional pavements. Permeable pavements, on the other hand, are intended to saturate the subgrade and compaction of a soil reduces the soil's hydraulic conductivity (Chapter 6).

The overall required pavement structure thickness will increase because of frequent subgrade saturated conditions and not compacting, or limiting the compaction of, the subgrade. The increased cross section most commonly shows up in the minimum required aggregate base course thickness.

Permeable pavement structure design must account for the hydrologic requirements (i.e. the thickness of the aggregate layer to temporarily store the stormwater runoff) and the requirements for the traffic loads.

10.4 Drainage Profiles

The term 'drainage profile' refers to how the different vertical layers are designed and assembled to drain and treat stormwater runoff. Given the flexibility of permeable pavement practices, a variety of drainage profiles could be designed based on design goals and objectives and site constraints. However, there are four basic drainage profiles commonly used:

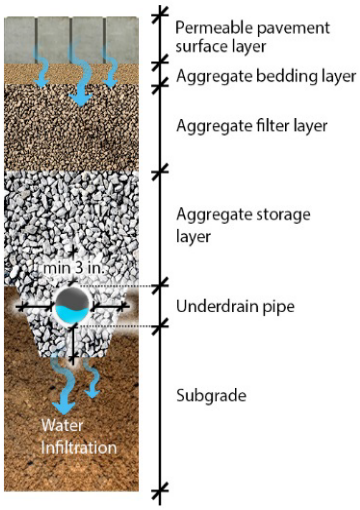

Figure 10-17 Typical conventional drainage profile

Conventional

These practices are designed to infiltrate as much water as they can into the surrounding native soils. A subsurface drainage system is incorporated into an aggregate storage layer to support partial infiltration and drain away any excess water from the profile. Water below the underdrain or subsurface drainage system outlet is forced to infiltrate. Conventional drainage profiles have:

- Permeable pavement surface and any required bedding
- Aggregate filter layers, as necessary
- Aggregate storage layer
- An underdrain system with a minimum of 3 inches of aggregate surrounding each underdrain on all sides
- A flat base below the lowest layer to maximize the area available for infiltration

Conventional drainage profiles work best when the infiltration rate of native soils is too low to satisfy the profile dewatering criterion, but infiltration is still desired. This profile helps to reestablish natural hydrologic processes.

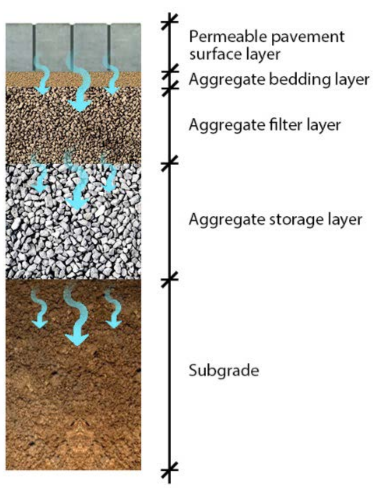

Figure 10-18 Typical pipeless drainage profile

Pipeless

Pipeless practices allow for infiltration of all water that enters the practice into surrounding native soils and are typically only used in areas with high permeability soil. No underdrains are provided. Pipeless profiles have:

- Permeable pavement surface and any required bedding
- Aggregate filter layers, as necessary
- Aggregate storage layer
- No subsurface drainage system
- A flat base below the lowest layer to maximize the area available for infiltration

Pipeless profiles work well for small drainage areas where the infiltration rate of the native soil is 0.5 inches/hour or greater and risk of clogging is low. This profile is effective at managing frequent small events and helps to reestablish natural hydrologic processes.

Sealed

Sealed practices do not allow for infiltration into native soils. All water that does not evaporate is conveyed through the subsurface drainage system in the aggregate storage layer. Sealed profiles have:

- Permeable pavement surface and any required bedding
- Aggregate filter layers, as necessary
- An underdrain system within an aggregate storage layer
- An impermeable liner surrounding the base and sides of the practice
- A slightly sloped base, or recessed underdrain that directs water toward the outlet of the practice

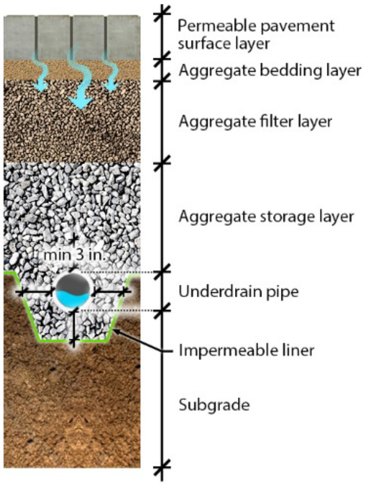

Figure 10-19 Typical sealed drainage profile

Sealed profiles are recommended when contaminated soils are found or are suspected near the practice, in the presence of karst geology, when the groundwater table is less than 2 feet from the bottom of the practice, when the practice is within 10 feet of a structure or foundation, or in any other situation where infiltration into the native soil is undesirable. These profiles retain the least amount of water of all the permeable pavement profiles and should only be implemented when site constraints dictate their use.

Steep Slopes

These practices are designed to infiltrate water when existing slopes prevent the use of a flat bottom across the entire practice. They are designed like a conventional profile with subsurface barriers constructed between terraced level bottoms to create separated infiltration beds. An underdrain may be used within an aggregate storage layer to drain away excess water. Water below the underdrain or subsurface drainage system outlet is forced to infiltrate. Steep slope drainage profiles have:

- Permeable pavement surface and any required bedding
- Aggregate filter layers, as necessary
- Aggregate storage layer
- An underdrain system (optional) with a minimum of 3 inches of aggregate surrounding each underdrain on all sides
- Subsurface barriers spaced to slow water and increase infiltration

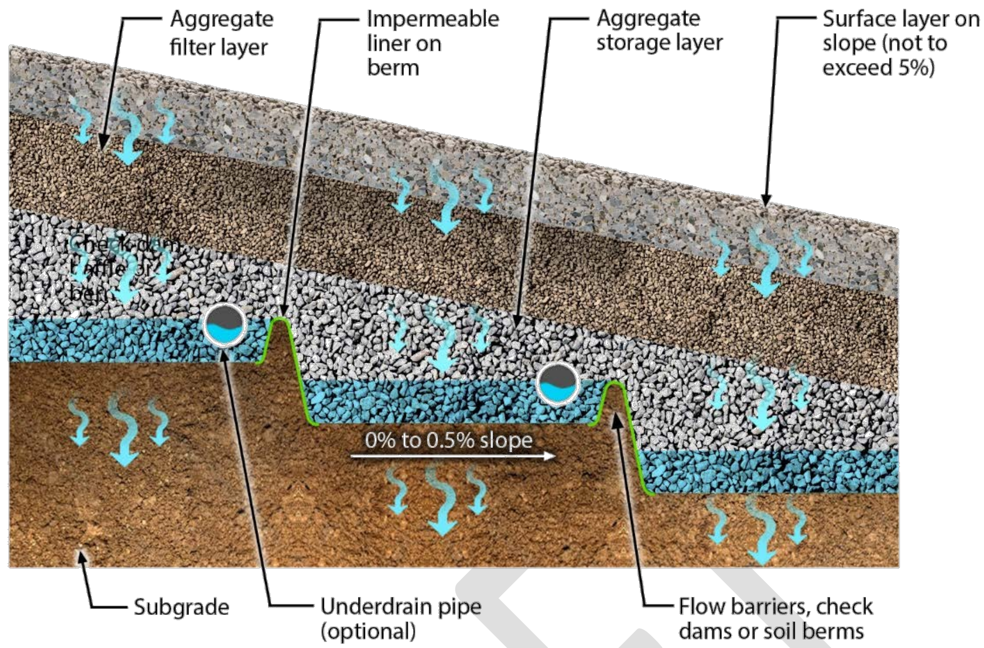

Figure 10-20 Steep slope drainage profile with subgrade terraces

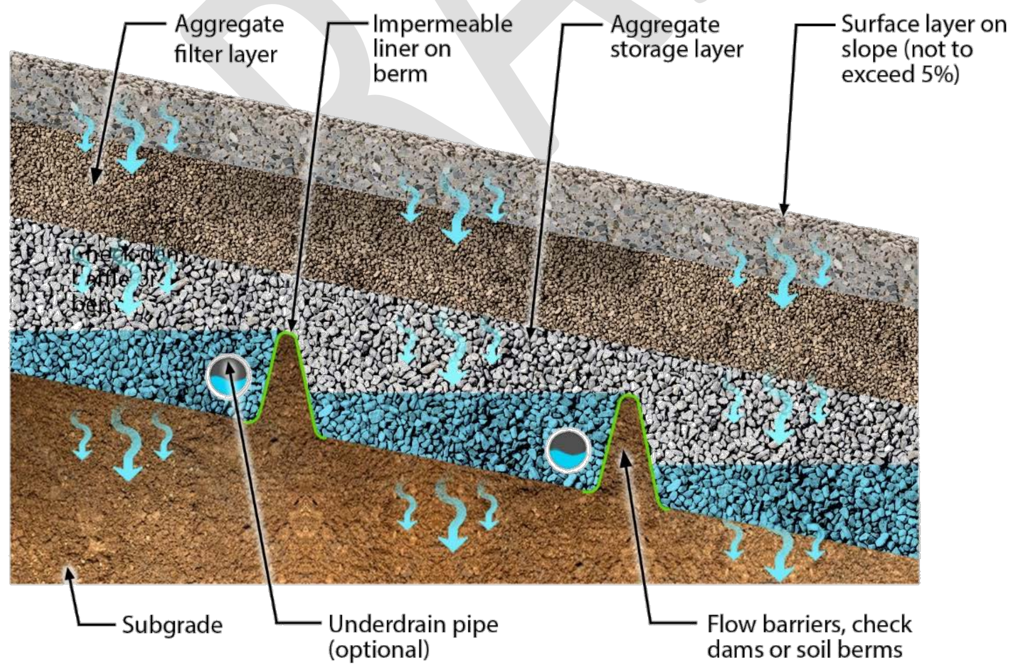

Figure 10-21 Steep slope drainage profile with sloping subgrade

Profile Selection

Drainage profile selection is governed by site constraints, design goals and objectives and any required performance standards. Most often, site constraints dictate which drainage profile can be used, and therefore, a straightforward process of elimination can help to determine the drainage profile best suited for a given site as illustrated in Figure 10-22.

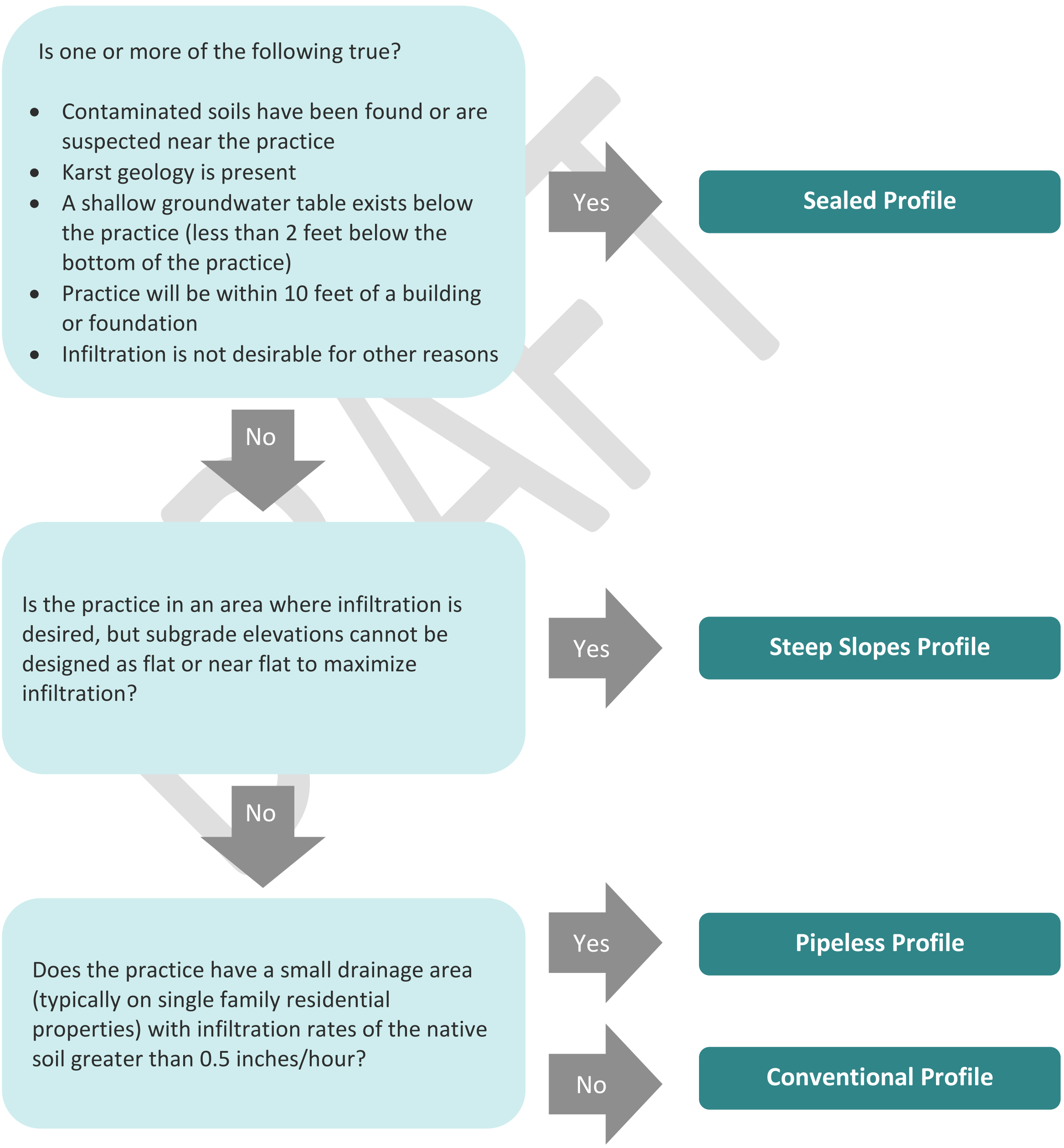

Figure 10-22 Drainage profile selection

10.5 Layers, Depths and Levels

Permeable pavements need to be designed to meet structural as well as stormwater management goals. The guidance and specifications detailed below are focused primarily on hydrologic function. Though there are many similarities among pavement types, each permeable surface option has unique structural, material and construction specifications critical to the long-term structural performance of the practice. Therefore, the designer should both follow pavement-specific guidance for the surface option chosen (NAPA, 2008; Smith, 2011; MCA, 2017; ACI, 2013) and specify minimum contractor qualifications for experience, training and/or certification.

The design of the selected drainage profile is an iterative process that works to eventually identify the appropriate depth of each layer based on site constraints and design goals of the project. Using this process, each of the following will be identified:

- Pavement surface elevations
- Aggregate filter layer depth and elevations of layer top and bottom
- Aggregate storage layer depth and elevations of layer top and bottom
- Internal water storage layer depth and elevations of layer top and bottom
- Subsurface drainage invert elevations
- Outlet invert elevations for all connections
- Overflow elevation

10.5.1 Sizing Profile Depth

Prior to sizing the area of the practice and calculating volumes, it is necessary to ensure that there is enough vertical depth to accommodate the selected hydrologic profile design. The process of sizing the profile depth is outlined below:

- Step 1: Determine maximum retention depth
- Step 2: Convert water depth to corresponding minimum profile layer depths
- Step 3: Add layer depths to determine the bottom elevation of the practice
    - Check against height of groundwater table
- Step 4: Determine invert elevation of subsurface drainage (if required)
- Step 5: Set outlet elevation
    - Check to see that positive drainage can be maintained from outlet to municipal sewer system connection
- Step 6: Iterate depths and levels until all design criteria have been met for retention
- Step 7: Determine detention depth available beyond retention depth (can include storage in permeable pavement surface layer only if product is designed for surface storage)
- Step 8: Set overflow elevation
- Step 9: Iterate depths and levels until all design criteria have been met for detention

Key Criteria:

Dewatering Duration

- Surface Dewatering: < 30 seconds
- Profile Dewatering: 72 hours

Infiltration Rate Safety Factor

0.50

*All measured infiltration rates shall be multiplied by a safety factor of 0.5 to obtain the **design infiltration rate**.*

Dewatering Criteria

The entire profile of the practice must be dewatered in 72 hours to restore hydraulic capacity to receive flows from subsequent storms and maintain infiltration rates. Surface ponding on permeable pavements is highly discouraged, therefore all the temporary storage for the stormwater runoff should occur under the pavement unless the surface product is specifically designed for water storage.

Retention Depths

To begin the process of sizing the profile depth, the *maximum retention depth* of the practice needs to be determined. Retention in permeable pavement practices is governed by the infiltration rate of the native soils following any required compaction and applicable dewatering criteria. The maximum retention depth is the maximum depth of water that can soak into the ground and still meet the allowable dewatering duration criteria (Equation (10.1)). The following subsections on the different layers of the profile describe how to use the *effective porosity* of the corresponding layer material to convert equivalent water depths into layer depth.

$$d_{max} = f_c * D_d \tag{10.1}$$

where d_{max} = maximum retention depth, in
f_c = design infiltration rate, in/hr
D_d = dewatering duration criteria, hr.

10.5.2 Permeable Pavement Surface Layer

General descriptions of several types of permeable pavement surface layers are provided in Section 10.1.2. For information regarding material availability, quality and sizing the depth of the layer, refer to the appropriate manufacturer's recommendations and their product representatives. Information regarding the most common permeable pavement surface types are discussed in greater detail below.

Porous Asphalt

Porous asphalt mixes are typically made with polymer-modified asphalt to improve the performance of the mix (NAPA, 2008). While not required, polymer-modified asphalts are recommended when available for most applications. Mix designs for porous asphalt pavements shall adhere to the most current recommendations from the Asphalt Paving Association of Michigan, the National Asphalt Pavement Association (NAPA), the American Association of State Highway and Transportation Officials (AASHTO), the

Federal Highway Administration (FHWA), and any state or local requirements governing the installation of porous asphalt.

Table 10-3 provides recommended AASHTO layer coefficients for the structural evaluation of porous asphalt and Table 10-4 provides recommended minimum thicknesses for the porous asphalt surface for different traffic loadings (NAPA, 2008).

Table 10-3 Recommended AASHTO Layer Coefficients for Porous Asphalt

Material	Layer Coefficient
Porous Asphalt	0.40-0.42
Asphalt Treated Permeable Base (ATPB)	0.30-0.35
Open-Graded Aggregate Base Layers	0.10-0.14

Table 10-4 Porous asphalt recommended minimum thicknesses

Traffic Loading	Minimum Compacted Thickness
Parking (little to no trucks)	2.5 in.
Residential Street (some trucks)	4.0 in.
Parking/Street (heavy trucks)	6.0 in.

Pervious Concrete

The surface layer for pervious concrete practices consists of a specially designed concrete mix that incorporates 15% to 25% interconnected void space to allow for the infiltration of water. The Michigan Concrete Association provides training and certification for both Ready Mixed Concrete suppliers, as well as engineers, architects, and landscape architects to correctly design and specify pervious concrete mixes (MCA, 2017).

Mix designs for pervious concrete pavements shall adhere to the most current recommendations from the Michigan Concrete Association (MCA), the American Concrete Institute (ACI), the American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), and any state or local requirements governing the installation of pervious concrete.

Permeable Interlocking Pavers

Typically, the design information required for the installation of permeable interlocking pavers can be obtained from the manufacturer and installed according to their specifications. Today, pavers come in a variety of shapes and sizes, and the designer should ensure that the selected pavers meet the necessary structural requirements for use in vehicular applications, and all ADA requirements for use in pedestrian applications (USDOJ, 2010). All concrete pavers for use in permeable pavement applications shall comply with the requirements of ASTM C936, Standard Specification for Solid Concrete Interlocking Paving Units. When used in vehicular traffic applications, they must also have an overall length-to-thickness ratio less than or equal to 3 with a minimum overall thickness of 3.125 inches (ASCE, 2016).

When using pavers in vehicular traffic applications, pavers shall be laid in a 45-degree or 90-degree herringbone pattern as shown in Figure 10-23. This pattern is especially important at intersections and in other areas where vehicles make turning movements to prevent the displacement of the pavers. A sailor or soldier course (Figure 10-23) against the edge restraint aids in structural integrity of the surface and is also recommended for increased stability at the edge (ASCE, 2016).

Key Criteria:

Pedestrian Areas (ADA Compliance)

- Changes in level: 1/4" max vertical change between pavers or adjacent surface; 1/2" max vertical change if edge is beveled at 2H:1V max
- Joints: spaces between pavers or adjacent surfaces must be less than 1/2"
- Detectable warning surfaces: required when accessible pedestrian routes enter vehicular traffic

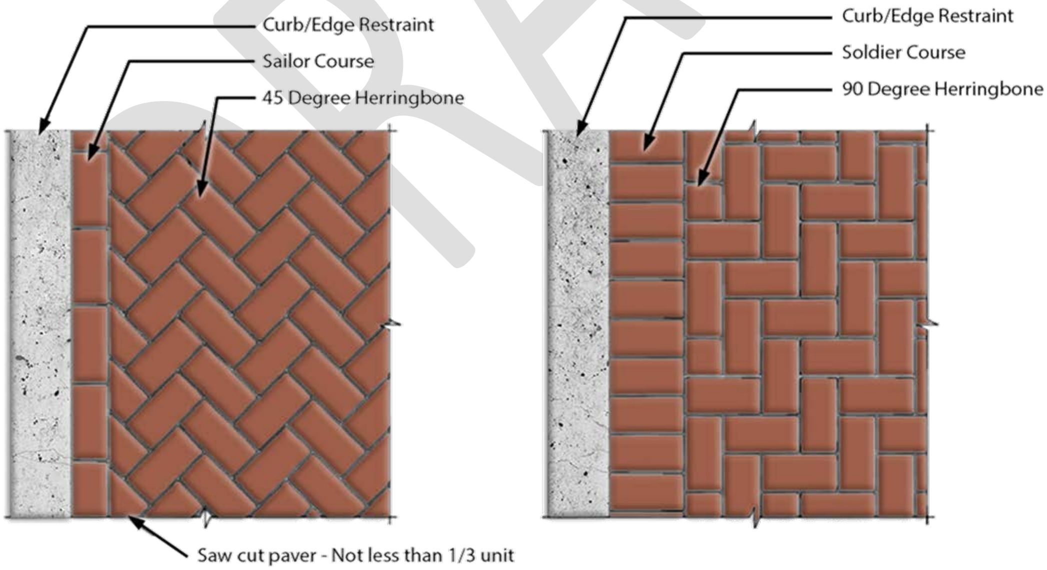

Figure 10-23 Herringbone pattern with single sailor course; herringbone pattern with a single soldier course

10.5.3 Aggregate Layers

Unless otherwise recommended by a pavement specifier's guide, all aggregates used for permeable pavement practices shall be clean, crushed/angled open-graded aggregate. Depending on the selection of the permeable pavement surface, the number of aggregate layers required for the practice could be anywhere from one to five separate layers. Additional guidance on the selection of aggregates beyond what is provided below can be found in Chapter 6, Soil, Aggregates and Water.

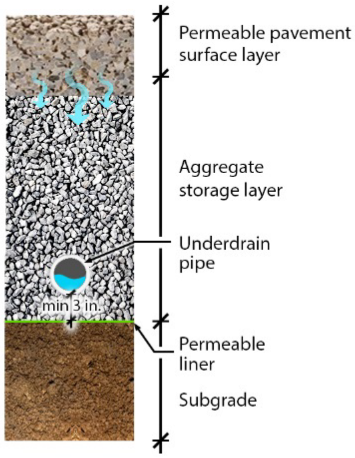

Figure 10-24 Single depth aggregate storage layer

Aggregate Storage Layer

Typical aggregate for the storage layer will be 1.5 to 3.0-inch diameter crushed aggregate (MDOT 4AA coarse aggregate or AASHTO #1 or #2 crushed limestone). A choker course (MDOT 6AA or AASHTO #57 crushed limestone) is typically required to level and "lock in" the surface of the storage layer before the pavement or bedding layer is installed. Appropriate compaction of the choker layer is especially important for "flexible" pavement surfaces such as porous asphalt or concrete pavers to prevent uneven settling of the pavement surface under differential traffic loads.

Calculating Aggregate Storage Layer Depth

The depth of the aggregate storage layer is dependent on the volume of water that is to be stored there. It is determined by converting the maximum equivalent water retention depth (Equation(10.1)) to a minimum layer depth (Equation (10.2)). Unless independent testing is conducted on the materials to be used, a value of 0.30 shall be used for the effective porosity of the coarse aggregate.

$$d_{agg} = d_{max} * \eta_{eA} \tag{10.2}$$

where d_{agg} = minimum depth of the aggregate layer, in.
d_{max} = maximum equivalent water retention depth, in.
η_{eA} = effective porosity of the aggregate, fraction

Effective Porosity of Aggregate

$$n_{eA} = 0.30$$

By comparing the bottom elevation of the aggregate storage layer to the elevation of the groundwater table, the depth of the aggregate storage layer can be adjusted to accommodate any additional detention storage that may be required for the site.

Aggregate Filter Layer

Oftentimes, when an open-graded aggregate storage layer is placed directly on top of a subgrade, or when an aggregate bedding layer is placed directly on top of an open-graded aggregate storage layer, the finer particles of the smaller aggregate or soil migrate into the much larger pore spaces of the larger aggregate. To prevent this from happening, both layers either need to be designed such that migration between the two layers does not happen, or an aggregate filter layer (also referred to as choker layer or transition layer) needs to be placed in between. Geotextile fabrics are often recommended between aggregate storage layers and subgrade, but have a high susceptibility for clogging as fine sediments migrate downward through the profile. Including an additional aggregate layer with particle sizes that are in between that of the subsoil and the aggregate storage layer is the best way to ensure permeability and prevent migration of fines moving into or through adjacent layers. Typically, this layer is designed with a thickness between 4 and 6 inches. Additional aggregate filter layer sizing criteria are included in Chapter 6, Soil, Aggregates and Water.

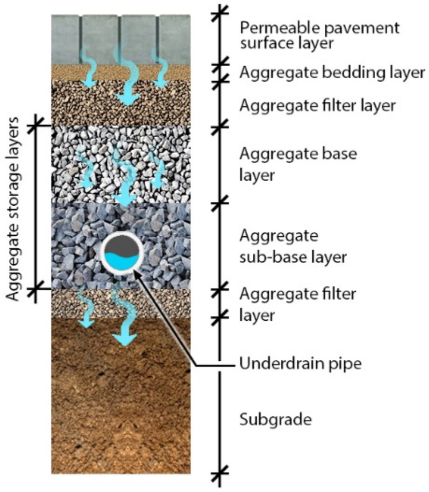

Figure 10-25 Five layers of aggregate storage based on aggregate sizing requirements

Aggregate Bedding Layer

When the permeable pavement surface layer consists of some sort of paver, an aggregate bedding layer is included to provide a smooth, even surface upon which to set the pavers. Refer to the paver manufacturer's recommendations for material selection and depth.

10.5.4 Permeable Pavement Layers Relative to Groundwater

When groundwater tables are too close to the bottom of an infiltrating practice, it can have potential negative effects. High seasonal or permanent groundwater tables can reduce the infiltration capacity of the practice, leach pollutants into the groundwater, or even lower groundwater levels and discharge groundwater back into the municipal sewer system.

Therefore, the distance from the elevation of the lowest infiltrating layer of the permeable pavement practice to the seasonal high groundwater level or bedrock is recommended to be four feet. Two feet is allowable, but may reduce the performance of the practice.

Key Criteria:
Depth to Groundwater
(from bottom of practice)

- Suggested Depth: 4 feet
- Minimum Depth: 2 feet

10.5.5 Liners

When the selected drainage profile or site conditions require the separation of materials, a variety of liners can be used. Liners can either be impermeable to impede the flow of water or permeable to allow water to pass through. Additional information regarding the selection of liners can be found in Chapter 6, Soil, Aggregates and Water.

Impermeable Liners

Impermeable liners are used as part of the sealed drainage profile when adverse site conditions are present or when close to buildings precludes the infiltration of water into native soils. All seals between liner sheets, seals at pipe connections, and perforations shall be watertight.

Permeable Liners

Permeable liners may be used when there is a desire to prevent materials from mixing but infiltration is still desired and feasible. Even though the use of permeable liners is frequently recommended to separate aggregate storage layers and the subgrade, their use is strongly discouraged in situations where fines may lead to clogging and adversely affect infiltration rates in the practice. Designing an appropriate aggregate filter layer to serve this purpose is the preferred method (Section 10.5.3).

10.6 Layout

The layout for permeable pavement practices needs to account for a variety of factors during the design process to ensure sufficient space is available. The section below details the most common layout consideration.

10.1.1 Location

Placement and integration with other site elements should be incorporated as early in the conceptual design process as possible to minimize any space conflicts and maximize the most suitable locations. Factors to consider when locating permeable pavement practices within the site include most suitable soils for infiltration practices, available space, elevation and location of proposed practice regarding how water will be routed to the permeable pavement practice, and maintenance access.

Listed below are additional requirements that must be addressed when finalizing locations of permeable pavement practices.

Pavement and Subgrade Slope

Permeable pavements are commonly designed with at least a 1% surface slope. Providing a slope on the pavement surface promotes drainage in case of clogging. Permeable pavers have been successfully installed with a 12% slope, however 5% or less is more common and recommended.

The bottom of the excavated bed should be level, or nearly level, for practices that are designed to infiltrate. When designing a practice for temporary storage, the bottom of the excavated bed shall be sloped at approximately 1% toward the outlet or a subsurface drainage system to promote drainage. Subgrade slopes of 3 percent or greater will require engineered flow barriers, check dams or soil berms to increase infiltration and slow the flow of water in the downgrade direction (Figure 10-21). A terraced subgrade is recommended to increase the surface area for infiltration (Figure 10-20).

Setback Requirements

Permeable pavement practices are typically designed to collect and infiltrate stormwater runoff. It is important to maintain minimum separation distances from infiltrating practices and building foundations to prevent saturated soils compromising the integrity of the foundation, or from water supply wells to minimize the chance of contamination. Refer to Table 10-5 for common setback distances (SEMCOG, 2008).

Table 10-5 Setback Distances

Setback from	Minimum distance (feet)
Property line	2
Building foundation[1]	10
Municipal sanitary or combined sewer[2]	10
Public water supply well[3]	50

1. Minimum with slopes directed away from building.
2. The Department may stipulate additional setback requirements from certain municipal sewers.
3. At least 200 feet from Type I or IIa wells, 75 feet from Type IIb and III wells (MDEQ Safe Drinking Water Act, PA 399).

Maintenance Access

Regular maintenance is a key component to ensuring the long-term functionality of permeable pavement practices. Typical maintenance activities will require adequate access to all applicable subsurface drainage system structures for inspection, routine maintenance and vacuuming procedures. A full list of maintenance activities is presented in Section 10.12. It is recommended that this section be reviewed to ensure that proper access is provided for their execution.

10.6.2 Sizing the Practice

Once suitable locations have been determined and drainage profiles have been selected and adequately sized for retention, the area of the practice footprint can be determined. The steps below detail a simplified approach to determining the area of the practice to meet the applicable regulatory requirements.

- Using Chapter 2, Regulatory Requirements, determine which regulatory requirements are applicable to the practice.
- Delineate the drainage area contributing to the practice.
- Calculate the runoff volume desired to be permanently retained for all infiltrating practices using the methods discussed in Chapter 4, Hydrologic Procedures.
- Divide the calculated runoff volume by the equivalent water depth stored in the practice including any surface storage calculated in Section 10.5 above to determine the area required for infiltration.
- The contributing drainage area to the permeable pavements should be limited to paved surfaces, to avoid sediment wash-on. Where run-on from pervious

Key Criteria:
Drainage Area to Practice Surface Area Ratio

- Recommended Ratio: 2:1
- Maximum Ratio: 5:1

areas is unavoidable, pretreatment (e.g. a gravel strip or sump) should be provided.

- External drainage area contributing runoff to the permeable pavement should generally not exceed twice the surface area of the permeable pavement (i.e. a recommended ratio of 2:1). The maximum allowed ratio is 5:1.
- Ensure that all setbacks and maintenance access requirements have been addressed.

10.7 Outlets

Since permeable pavement practices often have infiltration rates that far exceed that of native subsoils, it is often necessary to provide a suitable outlet for excess water that enters the practice. Outlets are typically tied to the subsurface drainage system when one is used, but can also be used to simply provide positive drainage from the aggregate storage layer. The outlet shall be designed to ensure that any water entering the surface of the permeable pavement practice is not forced back up through the layers, which can dislodge aggregates in lower layers.

10.8 Subsurface Drainage

A subsurface drainage system consisting of a network of underdrains is required for all permeable pavement practices that include a) impermeable liners, b) are placed on top of soils having infiltration rates less than 0.5 inches per hour, or c) when practices are within 50 feet of a steep (greater than 20%) or sensitive slope. All underdrains must meet the following minimum requirements:

- All pipes and fittings shall be Schedule 40 or SDR 35 smooth wall PVC pipe. Corrugated HDPE will not be allowed.
- All pipes shall have a minimum diameter of 6 inches.
- Bend fittings shall not exceed an angle of 45 degrees.
- Riser pipes, cleanouts, and all piping not located within the permeable pavement practice, or within 5 feet of a structure connection shall be solid walled pipe.
- Underdrain laterals shall be perforated with a minimum of 3 rows of 3/8-inch diameter perforations around the circumference of the pipe. Perforations shall be placed 6 inches on center within each row for the entire length of the pipe.
- Underdrain laterals shall be installed either within an aggregate layer (Figure 10-27), or bedded and covered within a gravel envelope (minimum 3” bedding, minimum 3” cover) to prevent migration of soil into the underdrain.
- To prevent clogging, underdrain pipes shall not be wrapped with a geotextile.
- A cleanout location shall be provided at the terminal ends of each underdrain.

10.8.1 Observation Wells

Design drawings should specify installation of observation wells to monitor the drawdown rate of permeable pavement reservoir layers. Wells should be constructed of perforated PVC pipe (4-inch diameter or greater) and should be designed to prevent damage from vehicular traffic. If necessary, observation wells can be installed at an angle and daylight in adjacent landscape areas (if the well extends the full depth of the reservoir layer). Wells should be securely sealed with watertight caps.

10.8.2 Subsurface Drainage Design

The subsurface drainage configuration greatly affects how water moves through the practice, the amount of water held for retention, and the hydrologic and water quality performance. Detailed design calculations for the subsurface drainage system are included in Chapter 5, Drainage Conveyance.

Conventional Drainage Profile

Conventionally drained practices feature underdrains that support partial infiltration by either elevating the underdrains within the aggregate storage layer, or placing them at the bottom of the aggregate storage layer and using an upturned elbow within an outlet or overflow structure to control the amount of infiltration (Figure 10-27). By placing the underdrains at the bottom of the aggregate storage layer and controlling the depth of infiltration with an upturned elbow, the height of the vertical pipe inside the outlet or overflow structure can be adjusted based on the performance of the practice over time. Furthermore, the capped tee-connection allows easy access to the subsurface drainage system for inspection and maintenance (Figure 10-26).

Figure 10-26 Upturned underdrain with a capped tee connection

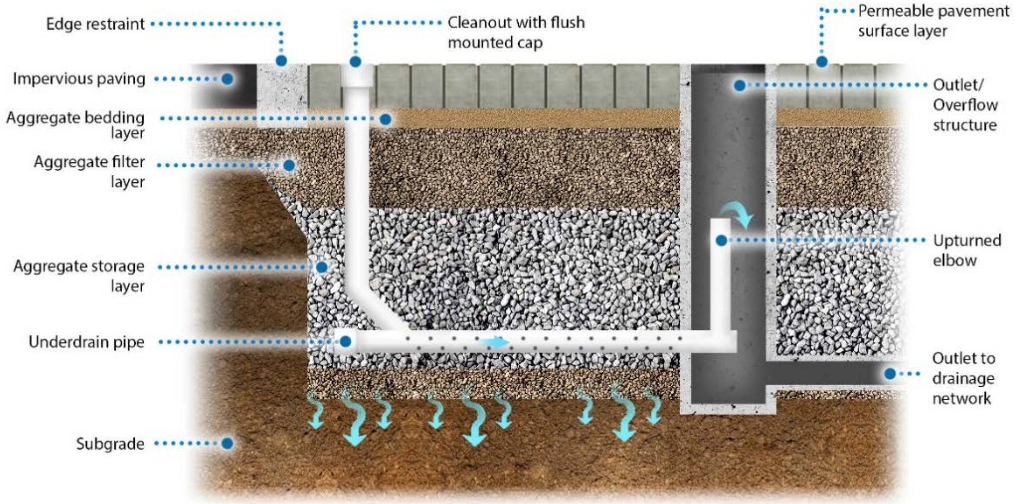

Figure 10-27 Conventional drainage profile with upturned elbow in outlet/overflow structure

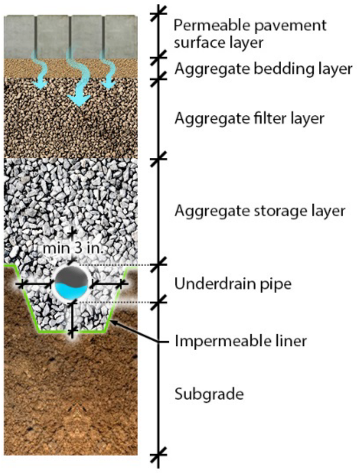

Figure 10-28 Sealed Drainage Profile

Sealed Drainage Profile

When the permeable pavement practice is lined with an impermeable layer, the underdrains are placed at the bottom of the practice (Figure 10-28). Typically, the bottom of the practice slopes slightly toward the underdrain, or each underdrain is recessed below the aggregate storage layer in an aggregate channel to remove any excess water from the practice.

10.8.3 Retention Volume

The volume of stormwater that can be retained in a permeable pavement practice changes dramatically when a subsurface drainage system is employed. In these practices, the volume retained is only the storage volume that is located below the subsurface drainage system outlet Figure 10-29.

$$V_R = V_A * n_{eA} \tag{10.3}$$

where V_R = retention volume, ft^3
V_A = volume of aggregate below subsurface drainage system outlet, ft^3
n_{eA} = effective porosity of aggregate, fraction

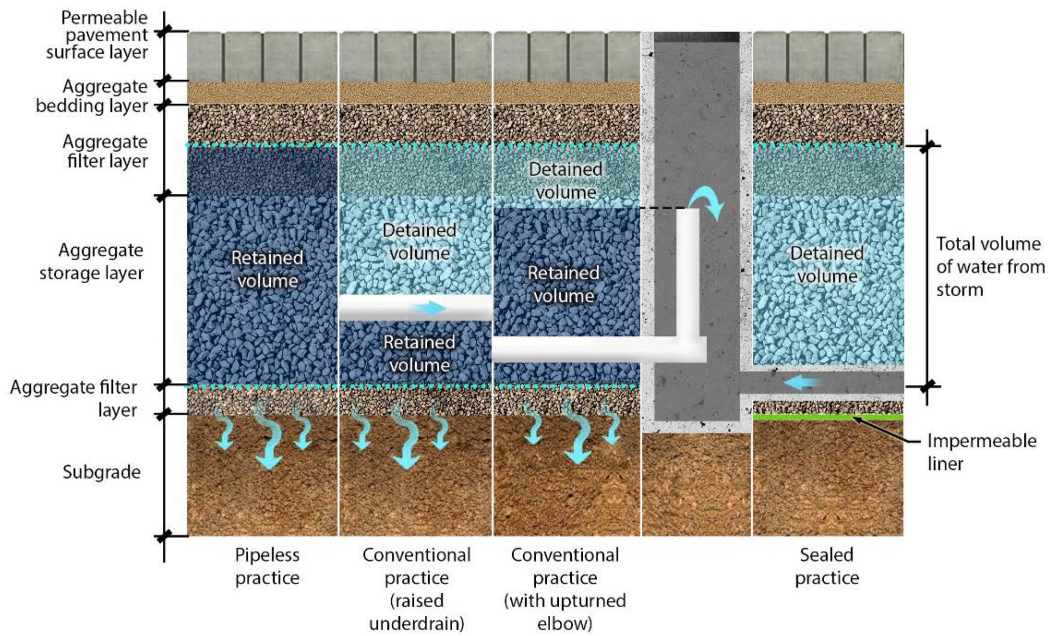

Figure 10-29 Cross-sections of drainage profiles

10.9 Overflow Conveyance

In addition to providing an outlet for excess stormwater that enters through the surface of the pavement, it is also necessary to provide an alternate path for excess runoff that exceeds infiltration rates of the permeable pavement surface layer. Events like this may occur during high-intensity rainfall events or if the surface becomes plugged (Dylla & Hansen, 2015). The figure below (Figure 10-30) illustrates two options that may be used for overflow conveyance.

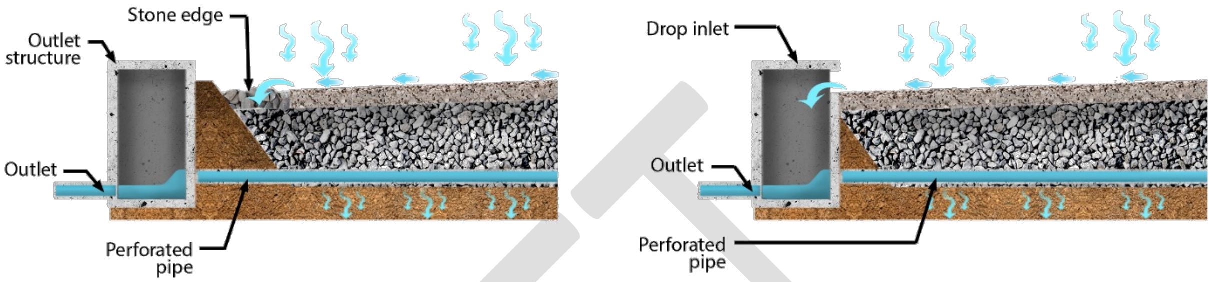

Figure 10-30 Permeable pavement overflow configurations

The first option uses a stone edge at the end of the pavement. The stone edge is designed such that water can pass through to the drainage system by using an open graded aggregate. One advantage of this design is the aggregate storage layer beneath the pavement is utilized regardless of if water passes through the pervious pavement. Alternatively, the pavement may be graded to discharge to another stormwater management practice. This option is most applicable when the pavement is designed without a curb.

The second option uses traditional catch basin inlets in the design. These may be combination inlets at the curb face or flat inlets located in the pavement. One disadvantage of this type of design is that if the pavement clogs and prevents infiltration, the stormwater runoff will bypass the aggregate storage layer and subgrade infiltration. Additional information on spacing and sizing of outlet/overflow structures can be found in Chapter 5, Drainage Conveyance.

Pervious concrete parking, overflow conveyance to bioretention

Permeable paver parking; overflow conveyance to flat catch basin inlet.

Pervious concrete parking; overflow conveyance to combination catch basin.

Figure 10-31 Overflow conveyance options

10.10 Edge Restraints

Providing separation between permeable pavements and adjacent impermeable surfaces serves multiple purposes, including the following:

- Clearly identifying for maintenance personnel, the transition between permeable and impermeable surfaces
- Restraining modular (block) pavers and porous asphalt to prevent lateral shifting or unraveling of edges
- Creating a hydraulic restriction layer to prevent lateral seepage of runoff below adjacent pavements and structures
- Delineating parking zones with clean, aesthetically pleasing lines

Figure 10-32 Concrete edge restraint between PICP and traditional asphalt

Restraints for flexible pavements are typically composed of standard concrete curbs (elevated or at grade, depending on application) or specially designed monolithic concrete walls. At intersections between permeable and impermeable surfaces, a hydraulic restriction layer (typically a geomembrane) is installed along the entire length of the cut and at least 2 feet laterally along the subgrade and under the impermeable surface. Figure 10-32 shows an example of a typical edge restraint.

10.11 Construction Considerations

Although installation practices of permeable pavements are like their conventional counterparts, proper installation during all phases of the construction ultimately determines if permeable pavements will function as intended. In addition to requiring that installation be performed only by contractors with experience and applicable certifications in permeable pavement installation, careful inspection of several pertinent construction steps can prevent costly errors.

Certifications are provided by the Interlocking Concrete Pavement Institute (http://www.icpi.org) and the National Ready-Mix Concrete Association (http://www.nrmca.org). Lists of recommended porous asphalt installers are provided by the Asphalt Pavement Association of Michigan (www.apa-mi.org).

10.11.1 Inspections and Testing

Frequent inspections by the designer or a trained inspector during construction and testing during critical installation steps can help to ensure the function of permeable pavement practices. Considerations for the following should be incorporated into the specifications and construction drawings for the project.

Soil Erosion and Sedimentation Control

Sediment entering a permeable pavement practice during construction can greatly diminish the performance of the practice. Sediment infiltration at any stage can reduce the permeability of the subgrade soil, clog the aggregate storage layer and filter layer, as well as the permeable pavement surface layer. Prior to construction activities beginning, all soil erosion and sedimentation controls shall be installed and then inspected regularly. Preventing and diverting sediment from entering the practice area during construction must be the highest priority. All controls shall be inspected regularly throughout construction, especially following rain events, to ensure their function. Best practices such as keeping muddy construction equipment away from the area, installing silt fences, staging excavation, and temporary drainage swales that divert runoff from the area will make the difference between a pavement that infiltrates well or poorly. Moreover, the pavement must not receive any runoff until the entire contributing drainage area has been stabilized.

Subgrade Preparation

Subgrade preparation is critical. The subgrade should be inspected and tested to ensure that it has been prepared according to specification regarding any slopes, design infiltration rates, and levels of compaction. If the subgrade is found to be compacted beyond specified levels, Chapter 6, Soil, Aggregates and Water offers some practical suggestions for mitigating the effects of compaction.

If subgrade exfiltration rates are substantially lower than original design rates, it may be necessary to provide additional aggregate reservoir depth to accommodate storage and exfiltration of subsequent rainfall events.

Materials Inspection

Aggregates - Stone aggregate bedding, filter, and storage layers should be thoroughly washed to prevent fines from clogging the subsoil interface or underdrains. Before placement, the furnished aggregate should appear free of fines and leave no substantial dust on the skin when handled. Unwashed aggregate should be replaced or washed onsite using proper construction site sediment control practices.

Elevations – Elevations should be verified following the installation of each layer within the profile to ensure that the correct slopes and depths have been adhered to during construction. Discrepancies during grading or placing of pipe inverts can result in undersized and underperforming systems that are significantly difficult to correct once installation has been completed. Furthermore, the slopes on any transition curbing or adjacent impermeable surfaces should be verified for positive drainage toward the permeable pavement practice.

Figure 10-33 Inspection of moisture content in pervious concrete mix during delivery

Surface Course Placement – Poured-in-place surface courses should be inspected during placement to ensure proper mix characteristics. After screeding and compaction, inspectors should ensure that the surface of pervious concrete is not smeared or sealed over in any areas (i.e. surface looks smooth like traditional concrete) due to incorrect placing and finishing, particularly when placing plastic over the surface for curing.

When pavers are used as the surface layer, they should be inspected for chips and crack prior to installation. Review the manufacturer's specifications regarding what level of defect is acceptable in the paver blocks.

Post-Construction Performance Verification

Following installation of any permeable pavement practice, post-construction verification of infiltration rates shall be submitted as part of the Post-Construction Stormwater Management Plan. Testing procedures shall follow either the manufacturer recommended testing procedures for any proprietary surface product, ASTM C1701-09 for permeable asphalt and concrete, or ASTM 1781-13 for permeable interlocking pavement systems.

10.11.2 Excavation Techniques

When excavating and preparing areas intended for permeable pavement, the contractor shall employ practices which do not compact the subgrade beyond the limits specified in the construction documents. The following practices may be considered:

- Protect areas from heavy equipment and vehicular traffic to avoid over-compaction and reduction of permeability of the subgrade soil.
- Where possible, excavate the area from the sides to minimize subgrade compaction.
- Avoid use of excavation equipment with narrow rubber tires to minimize subgrade compaction; tracked equipment is recommended.
- Stockpile excavated materials away from any existing or proposed permeable pavement surfaces to prevent sediment from washing into those areas.

Figure 10-34 Protection of subgrade during construction

10.11.3 Protection of Work

Throughout the duration of construction, all existing work must be protected from any run-on, sediments, or other debris which may result in clogging of the completed work. This includes protecting exposed subgrades as well for practices that intend to infiltrate into the surrounding subgrade. All water shall be diverted from the area of the proposed practice until construction is complete and the contributing watershed has been stabilized.

10.12 Operation and Maintenance

Maintenance of permeable pavement systems is critical to the overall and continued success of the system. Specific maintenance activities are listed in Table 10-6. Key maintenance procedures consist of the following:

1. Adjacent areas that drain to the permeable pavement area should be permanently stabilized and maintained to limit the sediment load to the system. Also, any use of salt or sand for de-icing and traction in the winter should be minimized.

2. Vacuum sweeping should be typically performed a minimum of twice a year. Adjust the frequency according to the intensity of use and deposition rate on the permeable pavement surface.

3. Any weeds that grow in the permeable pavement should be sprayed with pesticide immediately. Weeds should not be pulled, because doing so can damage the fill media.

Table 10-6 Permeable pavement inspection and maintenance tasks

Task	Frequency	Indicator maintenance is needed	Maintenance notes
Catchment inspection	Annual	Sediment accumulation on adjacent impervious surfaces or in voids/joints of permeable pavement	Stabilize any exposed soil and remove any accumulated sediment. Adjacent pervious areas might need to be graded to drain away from the pavement.
Miscellaneous upkeep	Weekly or biweekly during routine property maintenance	Trash, leaves, weeds, or other debris accumulated on permeable pavement surface	Immediately remove debris to prevent migration into permeable pavement voids. Identify source of debris and remedy problem to avoid future deposition.
Preventative vacuum/ regenerative air street sweeping	Twice a year (spring after snowmelt and autumn after leaves fall) in higher sediment areas	N/A	Pavement should be swept with a vacuum power or regenerative air street sweeper at least twice per year to maintain infiltration rates.
Replace fill materials	As needed	For paver systems, whenever void space between joints becomes apparent or after vacuum sweeping	Replace bedding fill material to keep fill level with the paver surface.
Restorative vacuum/ regenerative air street sweeping	As needed	Surface infiltration test indicates inferior performance or water is ponding on pavement surface during rainfall	Pavement should be swept with a vacuum power or regenerative air street sweeper to restore infiltration rates.

10.12.1 Winter Considerations

Winter maintenance practices for permeable pavements can differ significantly from those employed on traditional pavements. All maintenance personnel should be educated on the specific requirements for any permeable pavement practices that are installed. In general, permeable pavements perform quite well in winter conditions. The well-drained nature of the permeable pavement surface layer limits frost penetration, reduces the formation of black ice, and often require less plowing (Roseen, Ballestero, Houle, Briggs, & Houle, 2011; SEMCOG, 2008). The following table lists a few common winter maintenance requirements for typical permeable pavement surfaces.

Table 10-7 Typical winter maintenance considerations

DO:	DO NOT:
Do: Plow snow carefully by either setting the blade about 1/2 inch above the pavement surface or use a rubberized blade tip.	**Do Not:** Use abrasives such as sand or cinders on permeable pavement surfaces or on adjacent surfaces. These products speed clogging of the permeable pavement surface
Do: Pile snow in adjacent grassy areas so that sediments and pollutants in the snowmelt can be partially treated before they reach the permeable pavement.	**Do Not:** Use deicing salts on pervious concrete, pervious concrete pavers or grid pavement systems planted with vegetation. Deicing salts in all other applications should be used moderately and only when necessary.
Do: Follow all manufacturer recommendations for winter maintenance procedures.	**Do Not:** Use brush attachments for snow removal in paver applications that require aggregate in the joints or on grid pavement systems with loose aggregate or soil.

Figure 10-35 Signage to remind maintenance staff of the presence of permeable pavement

10.12.2 Signage

Signage (Figure 10-35) educates the public about the purpose of the permeable pavement practice and signage also reminds maintenance crews on the presence of permeable pavement and maintenance practices. Signage also prevents permeable pavement from being mistaken as impermeable and then paved over during repair.

10.13 Design Checklist

The design of permeable pavement practices typically involves several iterations, design of several individual components, and frequent modifications during the design process. To ensure that the practice has been properly designed, the following checklist shall be used upon completion of the design, but before construction drawings have been finalized. This checklist can then be included as part of the Post-Construction Stormwater Management Plan along with any required calculations to document the design.

Table 10-8 Design Checklist

Description	Design Requirement	Recommended Values	Design Value
Treatment			
Drainage area tributary to practice			acres
Peak flow rate requirement	See Chapter 2 for applicable performance standards		cft/sec
Retention volume requirement	See Chapter 2 for applicable performance standards		cft
Design Flows and Volumes			
Minor design storm volume entering practice			
Minor design storm peak flow rate entering system			
Major design storm volume entering practice			
Major design storm peak flow rate entering system			
Drainage Profile			
Design infiltration rate of native soils		Greater than 0.5 in/hr. for pipeless profiles	in/hr.
Permeable pavement surface material			
Design infiltration rate of permeable pavement surface material			in/hr.
Drainage profile type			

Description	Design Requirement	Recommended Values	Design Value
Depth of permeable pavement surface layer	Based on manufacturer's specifications for permeable interlocking concrete pavers, grid pavement systems, pervious pavers, and rubber overlay and pavers	Porous Asphalt: 3"-7" Pervious Concrete: Sidewalks: 4"-5" Parking lots: 6"-8" Heavy loads: 8"	in
Depth of aggregate bedding layer	Per manufacturer's recommendations		in
Depth of aggregate filter layer (s)			in
Depth of aggregate storage layer	Minimum of 8 inches for vehicle loading, minimum of 4 inches for pedestrian loading		in
Liner type (permeable, impermeable, none)			
Surface dewatering duration	Less than 30 seconds		hr.
Profile dewatering duration	Maximum of 72 hours		hr.
Practice Elevations			
Inlet elevation			ft.
Permeable pavement surface elevation	Level with or slightly below inlet elevation		ft.
Overflow rim elevation			ft.
Average underdrain invert elevation			ft.
Upturned underdrain elevation			ft.
Outlet invert elevation			ft.
Bottom of practice elevation	Minimum of 2 feet above groundwater table elevation (not applicable to practices that do not infiltrate)	Minimum of 4 feet above groundwater table elevation (not applicable to practices that do not infiltrate)	ft.
Groundwater table elevation below practice			ft.
Layout			
Area of practice			sft
Area of lowest horizontal surface area for infiltration	Do not count bermed areas in steep slope profiles		sft

Description	Design Requirement	Recommended Values	Design Value
Surface slope	Maximum of 5% in all directions, maximum of 2% longitudinal slope along ADA pedestrian routes	Flat	
Bottom slope	Less than 0.5%, can be terraced in steep slope profiles		H:V
Distance to nearest property line	Minimum of 2 feet		ft.
Distance to nearest building foundation	Minimum of 10 feet (not applicable to practices that do not infiltrate)		ft.
Distance to nearest private well (where applicable)	Minimum of 50 feet (not applicable to practices that do not infiltrate)		ft.
Distance to nearest water supply well (where applicable)	Minimum of 50 feet (not applicable to practices that do not infiltrate)		ft.
Pedestrian areas, maintenance access and safety have been addressed	Confirm all applicable ADA requirements have been met, confirm maintenance procedures can be performed		
Inlet Design			
Inlet type			
Width of opening for discrete inlets		Minimum of 12 inches for curb cuts	in
Method of pretreatment			
Peak flow velocity over surface of practice (minor storm)			ft./sec
Peak flow velocity over surface of practice (major storm)			ft./sec
Subsurface Drainage System Design			
Minimum underdrain pipe diameter	Minimum of 6 inches		in

Description	Design Requirement	Recommended Values	Design Value
Minimum dimension of aggregate surrounding pipes	Minimum of 3 inches		in
Subsurface drainage system capacity			cft/sec
Overflow Design			
Overflow type			
Overflow capacity			cft/sec
Outlet Design			
Outlet Type			
Outlet pipe size	Minimum of 6 inches		in
Outlet capacity			cft/sec

10.14 References

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11. Rainwater Harvesting

Rainwater harvesting is a stormwater management practice where runoff is captured and reused in various processes. This chapter presents the several types of storage systems and the associated design elements. Focusing mainly on cisterns, the chapter discusses sizing, placement, reuse considerations, and maintenance.

Contents

11.1 Introduction

Rainwater harvesting involves capturing runoff from rain events and the using the water for some intended purpose such as irrigation. The primary purpose of rainwater harvesting is to provide an alternative water supply. Harvested water is temporarily stored in tanks called cisterns. Figure 11-1 shows an example of a large commercial site with two (2) 35,000-gallon cisterns used for irrigation which saves the property owner approximately 1.5 million gallons of city water per year.

Figure 11-1 Cistern Example Used for Irrigation

From the perspective of managing stormwater runoff, the storage tanks (cisterns) only provide an effective function if the tanks are empty (or partially empty) before the next time it rains.

The International Green Construction Code (not currently adopted by the City) addresses rainwater collection and distribution systems. Example potential uses identified in the code are shown in Table 11-1.

Advantages

- Reduces potable water use and associated costs
- Flexible design option allows system to be built in a variety of locations
- Provides educational benefits when placed at public or highly visible sites
- Can reduce size of infiltration practices if used in sequence

Limitations

- Has strict inspection schedule due to maintenance needs not being visible
- Aboveground systems require draining before a freeze to prevent structural damage
- Limitations due to year-round rainfall vs year-round demand
- May be subject to additional City, State, and Federal code restrictions

Table 11-1 Example reuse applications

Examples of Uses without Treatment	Examples of Uses with Disinfection and Filtration
• Outdoor Irrigation • Decorative Fountains • Yard Hydrants • Industrial Processes (e.g. dust control, indoor hose bibs spray) • Vehicle washing • Outdoor hose bibs (not routed through building wall)	• Toilet Flushing • Urinal Flushing • Evaporative Cooling Tower Make-up • Trap Primers • Fire Suppression Systems • Clothes Washers • Outdoor Pools and Spas • Hose Bibs - Residential

Rain barrels are very small cisterns. Rain barrels commonly collect rain water from a residential roof gutter and downspout and most commonly use the collected water for irrigation needs. Rain barrels are commonly thought as holding less than 100 gallons of water.

Figure 11-2 Example Rain Barrel

Because of their small capacity, a single rain barrel provides very little benefit for managing the overall runoff from a typical site. To illustrate this point, consider a small residential house with a 1,000-square foot roof. Assuming a downspout in each corner of the house and the flow evenly split, a single 80-gallon rain barrel can collect ½-inch of rainfall. The water quality treatment volume is 1-inch of rainfall, therefore two (2) rain barrels would be needed in each corner of the house, for a total of eight (8) rain barrels. The homeowner would then need to use the water and drain the rain barrels within a few days after each rain event if the goal is to manage runoff from the site.

Rain barrels are available to be purchased from various vendors or may be constructed from a plastic bin or repurposed container. Installation and construction instructions are available from many sources.

This manual focuses on the design and installation of larger cisterns.

11.2 Codes and Design Standards

Rain barrels used for supplemental irrigation on single family residential parcels do not require City approval. All other rainwater harvesting systems must be approved by the Buildings, Safety Engineering and Environmental Department (BSEED).

The Michigan Plumbing Code (Incorporating the 2015 edition of the International Plumbing Code) discusses nonpotable water systems in Appendix A. The provisions provided in the appendix are utilized as minimum guidelines in engineered systems applications and are not part of the code. (International Code Council, Inc., 2015).

Some highlights of Appendix A include:

- Rainwater shall be collected only from above-ground impervious roofing surfaces constructed from approved materials. Collection of water from vehicular parking or pedestrian surfaces shall be prohibited except where the water is used exclusively for landscape irrigation.
- Downspouts and leaders shall be equipped with a debris excluder or equivalent device.
- Enough rainwater shall be diverted at the beginning of each rain event, and not allowed to enter the storage tank, to wash accumulated debris from the collection surface. The roof washer shall operate automatically.
- Collected rainwater shall be filtered as required for the intended end use.
- Storage tanks shall be located a minimum 5 ft. horizontal distance from lot line adjoining private lots.
- Storage tank outlets shall be located not less than 4 inches above the bottom of the storage tank and shall not skim water from the surface.
- The quality of the water for the intended application shall be verified.
- A detailed operations and maintenance manual shall be provided.

The American Rainwater Catchment Systems Association (ARCSA) and the American Society of Plumbing Engineers (ASPE), with sponsorship support from the International Association of Plumbing and Mechanical Officials (IAPMO) and NSF International have developed standards for rainwater harvesting systems. The standards are intended to be consistent with, and complimentary to, the plumbing code. However, designers/installers are advised to consult with BSEED regarding local conditions, requirements, and restrictions. The applicable plumbing engineering & design standards are:

- *ARCSA/ASPE/ANSI 63-2013: Rainwater Catchment Systems*. The scope of this Standard covers rainwater catchment systems that utilize the principle of collecting and using precipitation from a rooftop and other hard, impervious surfaces. This Standard does not apply to the collection of rainwater from vehicular parking or other similar surfaces. (ARCSA/ASPE/ANSI, 2013)
- *ARCSA/ASPE/ANSI 78-2015: Stormwater Harvesting System Design for Direct End-Use Applications*. This Standard may be used in tandem with ARCSA/ASPE/ANSI 63-2013. This Standard applies to harvesting stormwater at or below grade from storm drain pipes and systems, not from the soil matrix. This Standard applies, for example, to the collection of stormwater from the transportation grid (vehicular parking, driving, or other similar surfaces), elevated parking structures, surface public rights-of-way, and storm drain systems. (ARCSA/ASPE/ANSI, 2015)

11.3 Rainwater Harvesting Elements

All rainwater harvesting systems are composed of a catchment area, conveyance system, filter, storage tank, foundation, and distribution system.

Catchment Area. Most commonly a roof, the catchment area collects rainwater and directs it to the conveyance system. The type of roof (metal roofs have a high runoff coefficient) and slope (steeper roofs usually have less contaminants) will affect the amount and quality of rainwater harvested.

Conveyance System. A conveyance system of downspouts and gutters moves the rainwater from the catchment area to the storage tank. Typically, existing rooftop gutters are used with additional pipes which tap into the downspouts and direct water to the cistern.

Debris Excluder. Debris excluders remove debris such as leaves, twigs and smaller particulate matter from harvested rainwater. Debris excluders may include gutter screens to remove larger debris from the roof.

Roof Washer. Roof washer refers to diverting the first flush runoff from entering the storage tank. The intent is to wash the debris that has accumulated between rain events from the collection surface.

Storage Tank. Cisterns are available commercially in numerous sizes, shapes, and materials. Many are made to custom fit the available space and can be short and wide, tall and narrow, round, rectangular, and almost any size imaginable. They can be made from multiple materials but are most commonly constructed of plastic or metal.

Foundation. A foundation is needed for larger cisterns. The cistern must have a foundation of either gravel or concrete based on the weight at capacity.

Distribution System. A distribution system can include pipes connecting the cistern to a greywater system inside the building, an irrigation system on the property, or other use for the harvested rainwater.

11.4 Supply and Demand

An evaluation of a site for a potential rainwater harvesting system should consider:

- *Purpose:* How will the harvested rainwater be reused?
- *Demand:* How much harvested rainwater is needed?
- *Supply:* How much rainwater can be collected?

Rainwater collected in a rainwater harvesting system can be used in many different applications, including landscaping or vehicle/equipment washing. Rainwater harvesting can also be used for certain indoor applications. Indoor applications typically require additional treatment and additional piping to avoid cross-contamination with the potable water supply. When assessing the primary use of the harvested rainwater, the timing of the use should also be assessed; will the water be used immediately, or stored for use later?

Sizing considerations for cisterns include both the frequency and volume of water usage demand, if applicable, and the frequency and volume of supply. Cisterns may also be sized to provide detention storage and flow control along with water conservation as a hybrid system.

The simple rule for sizing a rainwater harvesting system is that the volume of water captured is greater than or equal to the volume of water reused. Therefore, to maximize the benefits of the water harvesting system, the potential supply and demand volumes need to be determined.

11.4.1 Demand

After deciding on the primary use of the harvested rainwater, the next step is to estimate how much rainwater is needed and when it is needed. For example, if irrigation is the primary use, estimate the volume of water needed for the longest duration with no rainfall. If the water will primarily be for indoor greywater use, estimate the total daily usage and if there will be extended periods of non-use such as building closures.

Demand may be determined either using historic records of actual use, or by estimating each use. According to the USGS, the domestic use of publicly supplied water for 2010 in Wayne County is 85 gallons per person per day (USGS, 2018). Domestic water use includes the indoor household water use (e.g. drinking, food preparation, bathing, washing clothes and dishes, and flushing toilets) along with outdoor use such as watering lawns and gardens. About 30% of the water used in the average American household goes to outdoor tasks such as watering lawns and gardens. Nonresidential water use varies significantly based on the facility. Table 11-2 provides some typical water usage rates.

Table 11-2 Typical Rates of Water Use

Device/appliance	Typical Flow Rates
Urinals	1.0 gallons per flush[1]
Water Closet (Toilets)	1.6 gallons per flush[1, 2]
Residential washing machine	20-50 gallons per load[3]
Sprinkler	1-3 gallons per minute[3]
Lawn sprinkler, 3,000 ft^2 lawn, 1 in/wk.	1500-1900 gallons per week[3]
Fire hose, 1.5 in, 0.5 in nozzle, 65 ft. head	35-40 gallons per minute[3]
Fire hose, home, 125 ft. head, ¾ in	8-12 gallons per minute[3]

1. (International Code Council, Inc., 2015)
2. Federal plumbing standards for toilets is 1.6 gallons per flush. Older toilets use up to 7 gallons of water with every flush.
3. (Metcalf & Eddy, 1991)

Lawn turf generally requires 0.5 to 1.5 inches of water per week. This includes water from both rainfall and irrigation. Light, frequent applications of water (0.1 to 0.2 inches) are recommended instead of heavy applications once a week. During periods of elevated temperatures coupled with full sun and high winds, lawns will require more water, (Lyman, Rieke, & Vargas, 2002).

Landscape irrigation demands can vary widely depending on the plant species and density along with soil conditions and local climate. There are several science-based methodologies for estimating landscape irrigation demand.

- ANSI/ASABE Standard S623.1. Determining Landscape Plant Water Demands. This methodology provides an estimate of plant water demands of permanently installed, non-production based, established landscape materials. The standard does not cover plants for sports fields, golf courses, or food production. (ANSI/ASABE, 2017)
- SLIDE – Simplified Landscape Irrigation Demand Estimation. This method groups individual plant species into general plant-type categories. Water requirements are then estimated by weighting the area devoted to each plant category. (Kjelgren, Beeson, Pittenger, & Montague, 2016)

11.4.2 Supply

In theory, approximately 0.62 gallons per square foot of collection surface per inch of rainfall can be collected. In practice, however, some rainwater is lost to first flush, evaporation, splash-out or overshoot from the gutters in hard rains. The amount of runoff available to capture is given by Equation (11.1). In this case the precipitation (P) in the equation represents the rainfall amount of a given storm event.

$$V = 0.62 * C * P * A \tag{11.1}$$

where V = available volume for captured (gallons)
0.62 = Unit conversion (gal/in/square ft.)
C = runoff coefficient (typically 0.9 to 0.95 for roofs)
P = precipitation (inches)
A = drainage area to cistern (square feet)

The precipitation (P) in the equation may also represent a longer duration, for example a month or year. In which case the volume (V) represents the runoff volume for the same duration.

Equation (11.1) may be modified to account for the loss of the roof washer water, i.e. the first flush water diverted away from the storage tank. This may be accomplished by either subtracting the quantity of rainfall being diverted Equation (11.2) or as a known volume of water diverted, Equation (11.3). Note, the precipitation value in these equations only represents a single rainfall event.

$$V = 0.62 * C * (P - P_d) * A \tag{11.2}$$

$$V = 0.62 * C * P * A - V_d \tag{11.3}$$

where V = available volume for captured (gallons)
V_d = volume of runoff diverted away from the storage tank (gallons)
0.62 = Unit conversion (gal/in/square ft.)
C = volumetric runoff coefficient (typically 0.9 to 0.95 for roofs)
P = precipitation (inches)
P_d = precipitation diverted away from the storage tank (inches)
A = drainage area to cistern (square feet)

When using Equation (11.2), suggested values for the precipitation diverted away from the storage tank (P_d) are provided in Table 11-3 (ARCSA/ASPE/ANSI, 2013).

Table 11-3 Estimated Roof Contamination Potential

High Contamination[1]	Medium Contamination	Low Contamination[2]
0.20 in.	0.08 in.	0.02 in.

1. High contamination is considered to have high content of organic debris from animal waste, adjacent trees, and/or airborne contamination.
2. Low contamination is considered to minimal nontoxic contamination.

Another approach to account for the loss of water due to first flush diversions and other miscellaneous losses is to adjust the runoff coefficient (C) in Equation (11.1). In this case, a *C* coefficient of 0.75 to 0.90 is appropriate. This is a simple approach when computing the supply monthly.

11.4.3 Storage Capacity

To ensure an adequate supply of water, the storage capacity of the water harvesting system must be sized to meet the water demand through the longest expected interval without rain (refer to Chapter 4, Hydrologic Procedures) or the user should plan for another water source.

Assumed Dry Duration

One simple approach for sizing the storage is to assume a duration without rainfall and calculate the total demand for that period.

Example: Simple Storage Tank Sizing.
Assume 1.4 inch per week (0.2 in/day) of irrigation water is needed for 5,000 sq. ft. of turfgrass. Assume 9 days of storage is desired which should be sufficient about 95% of the time (refer to Chapter 4, Hydrologic Procedures). Then the storage tank size is calculated as:

$$Storage\ Tank = \frac{0.2\ in}{day} * 9\ days * \frac{1\ ft}{12\ in} * 5000\ ft^2 * \frac{7.48\ gal}{ft^3} = 5{,}610\ gal$$

Round the storage tank volume to 5,600 gallons.

Table 11-4 provides reference information for the capacity of varying sizes of round cisterns.

Table 11-4 Round Cistern Capacity (gallons)

Height (ft.)	4-ft. Dia.	5-ft. Dia.	6-ft. Dia.	8-ft. Dia.	10-ft. Dia.	12-ft. Dia.	18-ft. Dia.
6	564	881	1,269	2,256	3,525	5,076	11,421
8	752	1,175	1,692	3,008	4,700	6,768	15,227
10	940	1,469	2,115	3,760	5,875	8,460	19,034
12	1,128	1,762	2,538	4,512	7,050	10,152	22,841
14	1,316	2,056	2,961	5,264	8,225	11,844	26,648
16	1,504	2,350	3,384	6,016	9,400	13,535	30,455
18	1,692	2,644	3,807	6,768	10,575	15,227	34,262
20	1,880	2,937	4,230	7,520	11,750	16,919	38,069

Monthly

Another method of sizing a water harvesting system is to calculate the monthly demand and supply for the system. Monthly demand will vary depending on the intended use of the harvested water. For example, if used for irrigation then demand will be near zero in winter. Refer to Chapter 4, Hydrologic Procedures, for monthly rainfall data.

Daily

The most precise way to size a storage tank is to consider daily supply and demand. For this method, historical daily rainfall records are used. This method requires the use of a spreadsheet or computer model to help with the calculations. Supply is calculated based on the daily rainfall and the drainage area. Demand is specified daily accounting for climatic conditions and seasonal variations. The storage tank size is adjusted until the desired result is achieved.

11.5 Design Components

11.5.1 Inlet

Inlets shall be designed such that runoff can enter the cistern without backflow onto the roofs based on a 100-year 24-hr rainfall event. Calculations shall consider minor losses through the piping and fittings.

Connections can be made through the top of the cistern or high up on the side of the tank. Inlet connections made through the top of the cisterns may include a basket filter as an inlet filter option. Inlet connections through the sides, with the proper gaskets, are recommended for ease of maintenance and access to the cistern. Inlets should introduce water into the tank minimizing turbulence.

Inlet connections can feature either dry conveyance or wet conveyance while pretreatment includes filters and first flush diverters. The following subsections describe each configuration.

Dry Conveyance

When downspouts freely drain to the cistern without trapped water, the system is referred to as dry conveyance. Figure 11-3 illustrates a typical dry conveyance inlet configuration.

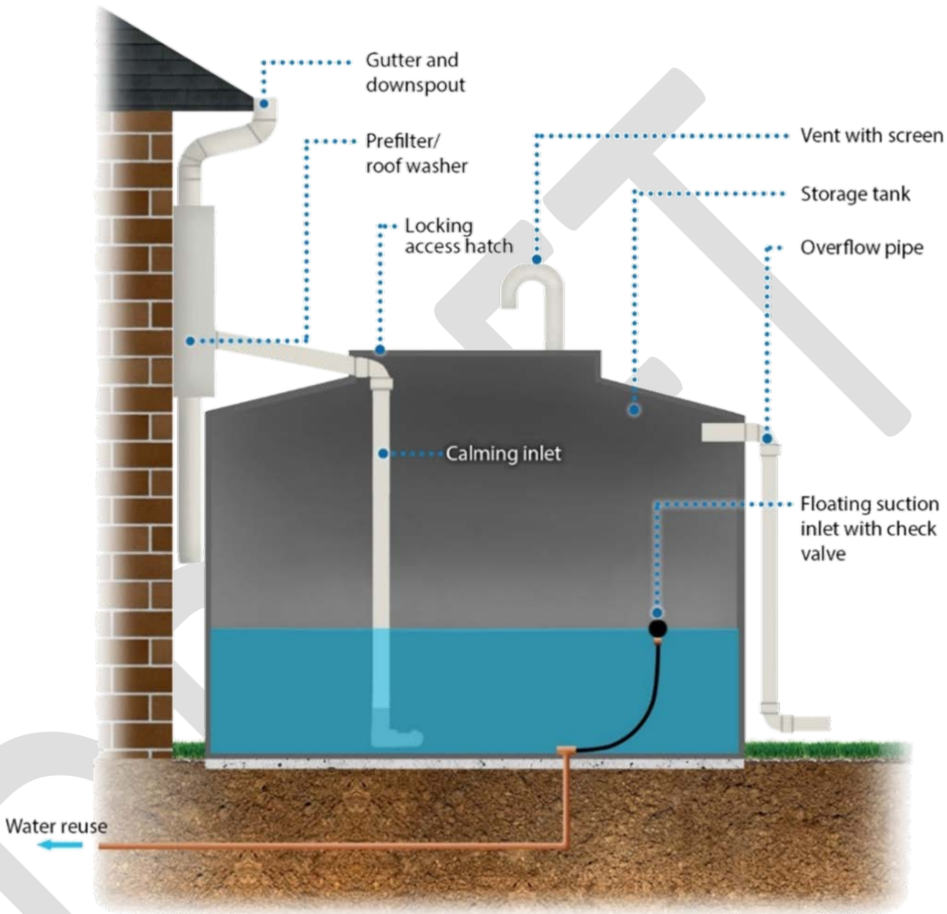

Figure 11-3 Dry conveyance inlet configuration

Wet Conveyance

When the downspout features a bend, causing water to be trapped between runoff events, this system is known as wet conveyance (Figure 11-4). Wet conveyance systems with buried downspouts allow cisterns to be placed further from buildings and might be preferable for aesthetic or overhead clearance purposes. Because water will permanently be stored in the downspout, watertight connections must be used to prevent leakage. A drain at the lowest elevation of the downspout should be installed for dewatering and emergency maintenance.

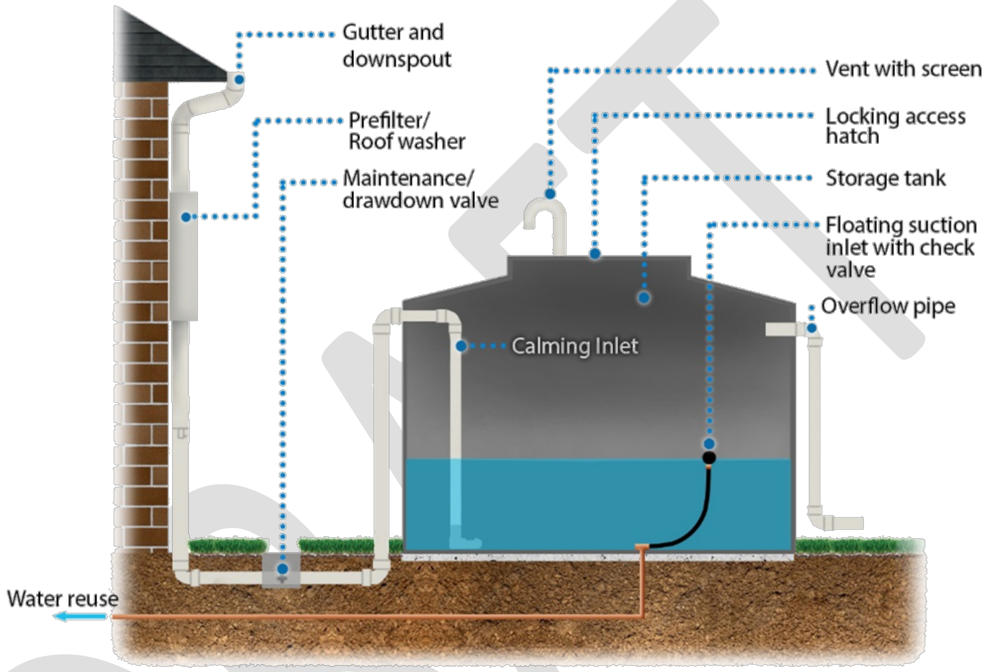

Figure 11-4 Cistern with wet conveyance featuring a drawdown valve for maintenance

11.5.2 Pretreatment

Stormwater runoff must be filtered before it enters the cistern to remove debris and particles that could clog the outlet. Two types of systems can be used: inlet filters and first-flush diverters. The following subsections discuss each pretreatment configuration in greater detail.

Inlet Filters

Inlet filters are designed to remove particles as runoff passes through the filters before entering the cistern; many filter options are available. The size and type of filter used will depend on the size of the area draining to the downspout. The filters can be installed at the gutter or at the end of the downspout depending on the configuration of the downspouts. Flow through filters that force all the runoff through the filter can be used for smaller drainage areas (less than 1,500 square feet). Filters capable of bypassing larger event flow could be required for larger drainage areas (1,500 to 3,000 square feet).

Debris screens should have openings no larger than 1/16 inch (0.15 cm). A self-cleaning screen used for inlet filters should provide a minimum angle of declination of at least 45 degrees from horizontal, but angles of more than 45 degrees tend to enhance self-cleaning and prevent clogging (Nel 1996).

Figure 10-5 Filter at the Inlet on a Cistern

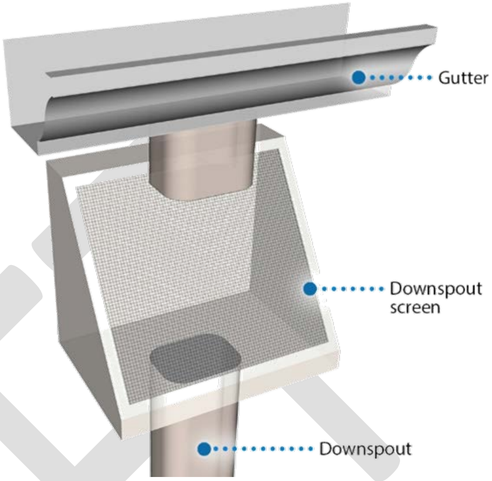

Figure 11-5 Debris Excluder

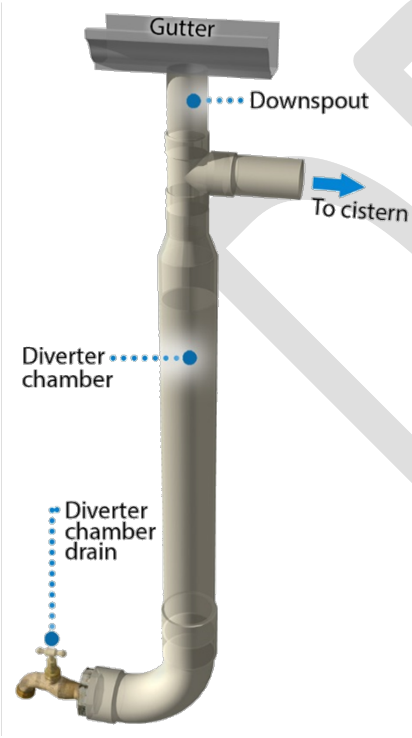

Figure 11-6 First-Flush Diverter

First-Flush Diverter

First-flush diverters can be installed after the inlet filter and are designed to divert an initial volume of water away from the cistern to prevent small particles—initially washed off the roof—from clogging the outlet. First-flush diverters are typically attached to the inlet or, in some cases, the inlet filter with a 4-to 6-inch diameter pipe with a small relief valve from which water can be diverted. The volume of water diverted away from the cistern depends on the size of the pipe. Once the diverter is full, water flows into the cistern. The volume captured by the diverter chamber is based on the rainfall depth from Table 11-3.

A first-flush diverter is not always required and inclusion is up to the designer depending on site conditions. A first-flush diverter is recommended for sites where pollen or other fine particles might not be removed by an inlet filter. Diverters must be routinely drained to provide capacity for the next runoff event. One approach is to place a drain valve at the end of the diverter and leave it slightly open to allow water to drain between events. Sediments captured in the diverter chamber may clog the small outlet, therefore the system needs to be checked regularly.

11.5.3 Storage Tank

Cisterns must also be watertight and must be sealed using a water-safe, non-toxic substance. Cisterns may be placed above- or belowground.

Aboveground

Aboveground cisterns are often less expensive to construct and operate compared to belowground systems. When the cisterns are elevated, water can be discharged by gravity, i.e. a pump is not needed. Aboveground cisterns should be protected from direct sunlight by positioning, landscaping or constructing a façade around the tank. Preventing direct sunlight on the tank helps to minimize algae growth.

Due to wintertime freezing temperatures, aboveground cisterns often need to be drained and not used during the winter months. This arrangement typically works fine if the harvested water is only used for irrigation and decorative fountains. Aboveground cisterns may be year-round if steps are taken to ensure the water in the tank does not freeze. One approach is to use an aerator to produce a steady stream of air that keeps the water moving. Another approach is to use a heat pump that recirculates hot water in the system.

Figure 10-7 Cisterns with wood facade

Belowground

When harvested water is needed year-round, belowground cisterns are more commonly used (Figure 11-7). They also offer the benefit of a relatively constant water temperature throughout the year, which is beneficial for year-round greywater reuse. The bottom of belowground tanks needs to be below the frost line along with the utility piping. Another advantage of a belowground cisterns is that they may be placed in the basement of buildings or under other infrastructure such as sidewalks and parking lots. Subsurface system tends to cost more and almost certainly require a pump to reuse the water.

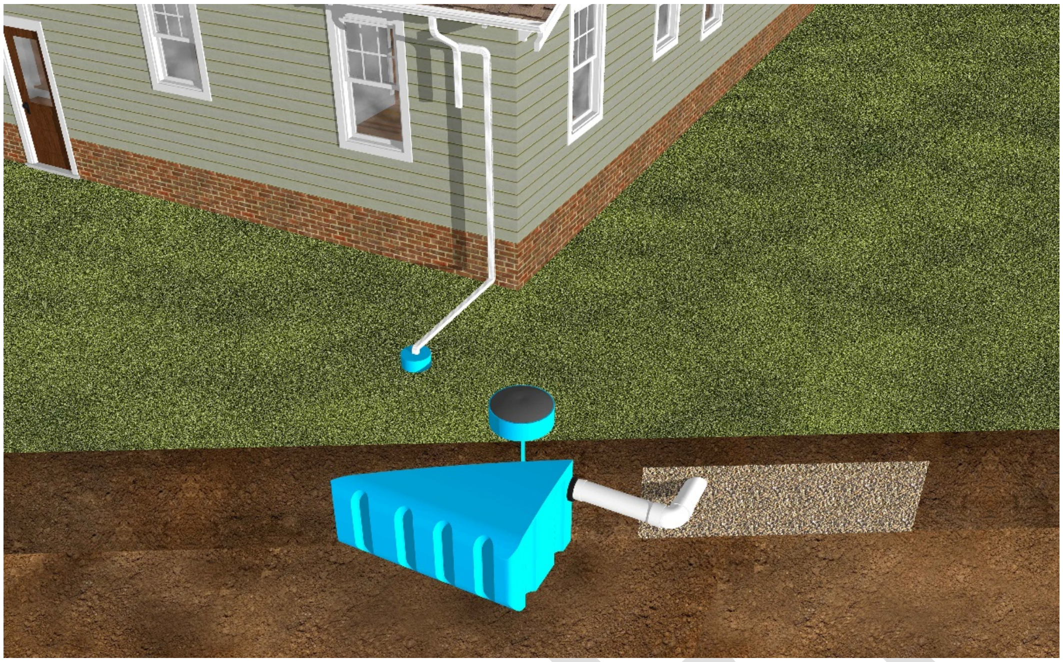

Figure 11-7. Belowground Cistern Illustration

Placement

To reduce the distance that water is conveyed, the tank should be placed as close as possible to the supply and demand points. Multiple cisterns placed around a structure can be hydraulically connected to take advantage of maximum storage capacity.

Material

Once the placement of the system is determined, the material and size of the tank should be considered. The tank itself may be made of many varied materials such as fiberglass, concrete, plastic, brick, and corrugated metal. When choosing size and material, the key things to consider are lifespan, foundation, cost, and aesthetics.

Structural Considerations

An appropriate foundation based on the weight of the tank when full of water is required to be designed by a structural engineer. Belowground storage tanks should not be buried below the water table unless an adequate foundation drain is provided and the design accounts for the buoyant forces when the tank is empty.

11.5.4 Outlet

Overflow or Bypass

All cisterns shall have an overflow for runoff volumes that exceed the capacity of the cistern. All connections shall be watertight. All overflows and water from the primary outlets shall be directed safely away from structural foundations and areas where infiltration could have an adverse effect. Overflow and bypass mechanisms shall be sized to safely convey the 100-yr discharge without any backwater onto the adjacent roof. Calculation of the 100-yr conveyance shall account for head losses through all pipe sections, elbows, entrances, and exits. Overflow discharges must be a minimum of 10 feet from any building foundation.

Low Flow Outlet – Passive Discharge

Cisterns may be designed to provide some detention storage in addition to the captured water for reuse purposes. This is accomplished by providing a passive outlet located part way up the tank wall. The passive outlet is left open to slowly drain the tank. The temporary storage above the passive outlet is the detention portion and the storage below the passive outlet is retained water for reuse (Figure 11-8).

Water discharging through the low flow passive outlet should be directed to vegetated area. The outlet's orifice should be designed to dewater the detained water nominally within 3 days. For passive discharge the orifice will often be very small and therefore should be equipped with a filter inside the tank to prevent clogging.

Refer to Chapter 7, Detention Practices, for additional information on designing detention storage systems.

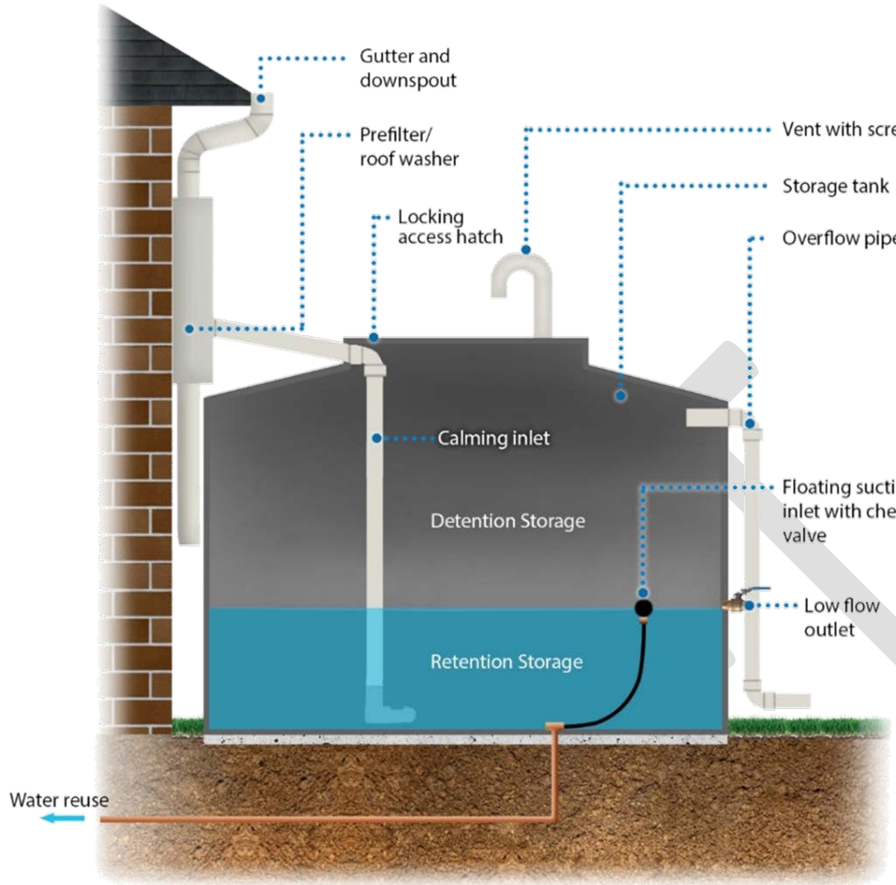

Figure 11-8 Cistern for detention and retention storage

Routing Water for Use

The method of routing water depends on the intended use. For basic irrigation, gravity can often be used to route harvested rainwater to nearby vegetation beds or infiltrating stormwater practices. To route water for use inside nonresidential structures or for greater distances from the cistern, a pump may be required. Submersible water pumps are commonly used, but pumps can also be installed in utility boxes next to the cistern.

Cisterns used to supplement greywater needs must install a parallel conveyance system to separate reused stormwater (or greywater) from other potable water piping systems. The greywater system must not be connected to domestic or commercial potable water systems.

11.5.5 Other Design Issues

Signage

Discharge points and storage units shall have signs indicating "Caution: Untreated Rainwater, Do Not Drink". These signs should be clearly visible and be placed near all outlets.

All pipes conveying harvested rainwater should be distinguishably colored to indicate that it is untreated water.

Figure 11-9 Signage Example

Vector Control

The inlets and outlets of cisterns and rain barrels shall discourage mosquito breeding. A piece of filter material, such as a screen or wire mesh may be used. A 1.2 mm or smaller mesh is recommended. Screens at the inlet should be placed downstream of debris filters to prevent clogging by leaves. Overflow/bypass openings shall be covered with a non-clogging configuration, such as a screen mesh flap that hangs across the pipe opening—the bottom of the flap shall be weighted or attached with small magnets such that it remains closed when no flow is present, but can easily open to allow overflow when the tank is full.

Makeup Water Supply for Dedicated Use

If the cistern will be used to offset non-potable water demand of nonresidential buildings (such as for toilet flushing) a makeup, or backup, water supply system is typically provided to maintain a minimum volume of storage water in the cistern for dedicated use. Several different makeup systems are available, most of which use floats and valves like toilet tank components. When the cistern level drops below the minimum capacity, the valve is opened and municipal water supply is used to fill the tank to a specified level.

11.6 Operation and Maintenance

Cisterns require regular maintenance during the rainy season to ensure proper function. Table 11-5 lists specific tasks which are described below:

1) The main source of debris in the cistern is leaf litter and other detritus collected in the gutter system. The gutter systems should be inspected and cleaned. Any leaks should be immediately repaired.
2) Check inlet filters to prevent clogging and debris accumulation to allow for proper flow into the cisterns. Clean as needed to ensure proper operation.
3) Outlet pipes and fittings should be inspected to verify proper flows from the cistern. Cisterns should empty within 24 to 48 hours.
4) Overflow systems should direct water away from any structural foundations.
5) Cisterns should be checked for structural stability and secured as necessary.

6) It is possible for some sediment and debris to accumulate in the bottom of the cistern. Access to the cistern should be maintained, and it is necessary to conduct a visual inspection to verify debris in the cistern.
7) Winterize the system not intended for year-round operation.
    a. Empty all water from the tank
    b. Empty water from the conveyance piping. Use compressed air if necessary.
    c. Drain the pump body completely. Store pumps in a climate-controlled location protected from the weather.
    d. Stop diverting water toward the tank.
    e. Leave drain valves open.

Table 11-5 Inspection and Maintenance Tasks for Cisterns

Task	Frequency	Indicator maintenance is needed	Maintenance notes
Drain flow diverter	After each rainfall	Diverter drain clogged with debris	Drain water from the diverter and remove foreign material clogging the drain valve.
Gutter and rooftop inspection	Biannually and before heavy rains	Inlet clogged with debris	Clean gutters and roof of debris that have accumulated, check for leaks
Remove accumulated debris	Monthly	Inlet clogged with debris	Clean debris screen to allow unobstructed stormwater flow into the cistern
Clean First Flush Diverter	Monthly	Filter is clogged with debris	Take diverter apart and clean each component.
Structure inspection	Biannually	Cistern leaning or soils slumping/eroding	Check cistern for stability, anchor system if necessary
Structure inspection	Annually	Leaks	Check pipe, valve connections, and backflow preventers for leaks
Overflow Inspection	Annually	Overflow clogged with debris	Check that overflow and outlets are clear of debris and directed away from building foundations
Disinfect cistern	Biannually	Odor or film on inside surfaces	Brush inside sufaces if possible and throughly disinfect
Drain cistern, pump and piping.	Before winter	Below freezing temperatures	To avoid damage, drain prior to winter so water does not freeze in cistern. Divert water away from cistern for the winter. Belowground cisterns often do not require winterizing.
Flush cistern	Annually	Excessive sediment	Flush to remove any sediment

11.7 Design Checklist

Table 10-5 Design Review and Checklist

Description	Yes	No	Notes
Define capture area and calculate supply			
Water use identifed and demand calculated.			
Draw down time conisdered?			
Size cistern based on supply and demand.			
Determine type of inlet: dry or wet conyenance			
Pretreatment provided to prevent debris from entering tank			
Is storage tank located for convienent use?			
Does the system have an acceptable overflow?			
Is a pump required?			
Has winter operation been considered?			
Has a maintenance plan been developed?			
Is a make up water supply needed?			

11.8 References

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12. Living Roofs and Walls

Living roofs and walls utilize vegetation and a light soil media to capture stormwater runoff on buildings. In addition to reducing the stormwater runoff from a site, the use of living roofs and walls can extend the life of building materials, reduce building energy demands by lowering indoor temperatures in the summertime, increase urban biodiversity, and even provide outdoor entertaining and relaxation areas. Both living roofs and walls are extremely versatile and can be configured to meet the structural needs of a building. These practices can be used to cover the entire available surface of a building or just a portion depending on the goals of the project.

Contents

12.1 Introduction

Advantages

- Reduces stormwater volume and peak flow through evapotranspiration
- Manages stormwater runoff without occupying surface-level space
- Well-suited for sites with roofs making up a large fraction of the site's imperious area, or for sites with limited ground-level space
- Can be used to reduce the size of other site stormwater control measures
- Reduces building energy use and energy costs
- Enhances roof lifespan

Limitations

- Roof structural constraints could preclude use
- Installation can be challenging in certain locations
- May require irrigation for maintenance of vegetation in summer months

Living roofs and walls have been used for centuries to provide protection from the elements and as places to grow food. While the original technology was not suitable for modern buildings, Germany has been experimenting with living roofs for decades and has led the effort in producing guidelines and regulations for their construction (FLL, 2008). Additionally, systems for living wall construction have been around almost as long (Green Roofs for Healthy Cities, 2008).

Living roofs, also called green roofs or vegetated roofs (Figure 12-1), reduce runoff volume and rates by intercepting rainfall in a layer of rooftop growing media. Rainwater captured in rooftop media evaporates or is transpired by plants back into the atmosphere. Rainwater more than the media capacity is detained in a drainage layer before flowing to roof drains and downspouts. Living roofs are highly effective at reducing rooftop runoff from small to medium storm events. Living roofs offer an array of benefits, including extended roof lifespan (due to additional sealing, liners, and insulation), improved building insulation and energy use, reduction of urban heat island effects, opportunities for recreation and rooftop gardening, noise attenuation, air quality improvement, bird and insect habitat, and aesthetics (Berndtsson, 2010; Getter & Rowe, 2006). Living roofs can be designed as extensive, shallow-media systems or intensive, deep-media systems depending on the design goals, roof structural capacity, and available funding. To improve vegetation survivability, a biodiverse, locally-adapted plant palette should be used. Even with careful plant selection, many "green" roofs will remain brown and dormant during parts of the year.

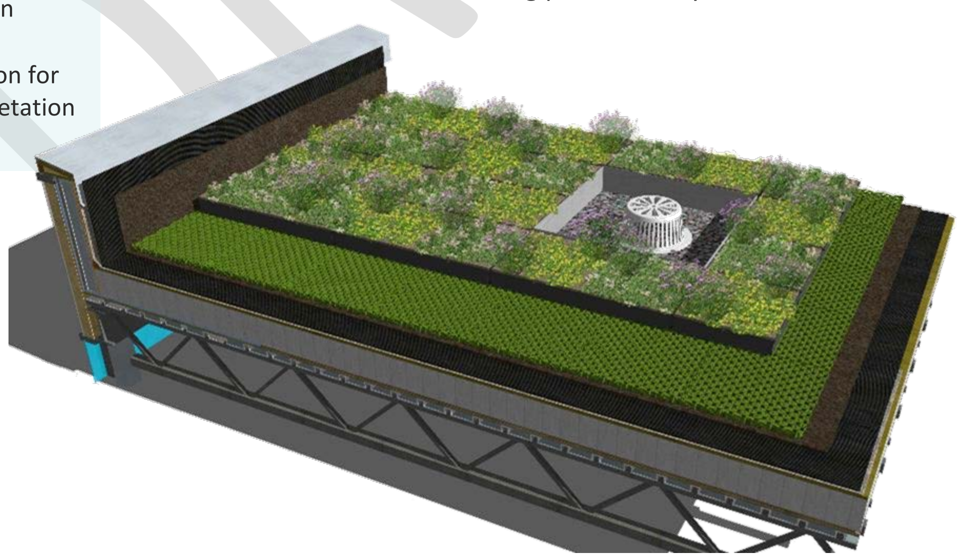

Figure 12-1 Typical living roof

Living walls, also known as green walls or vertical gardens, are comprised of plants grown in supported vertical systems that are generally attached to an internal or external wall. Similar to a living roof, living walls include vegetation, growing media, irrigation and drainage. However, living walls can vary greatly in their design and construction from simple plantings along a wall to modular and custom-fitted systems (Figure 12-2). Typical configurations for living walls include green façades, modular living walls and landscape living walls.

Living walls provide many benefits such as aesthetic improvements, reduction of the urban heat island effect, improvements in air quality, and improvements in building energy efficiency. However, living walls do not provide significant reductions in stormwater volume and require irrigation to support the vegetation. Cisterns can be placed higher than the top of the growing medium to help provide a constant supply of water.

Additional information and design recommendations for living roofs and walls can be found in *Guidelines for the Design and Construction of Stormwater Management Systems* (NYC DEP, 2012), Green Roofs for Healthy Cities (www.greenroofs.org), and *Growing Green Guide: A Guide to Green Roofs, Walls and Façades in Melbourne and Victoria, Australia* (www.growinggreenguide.org).

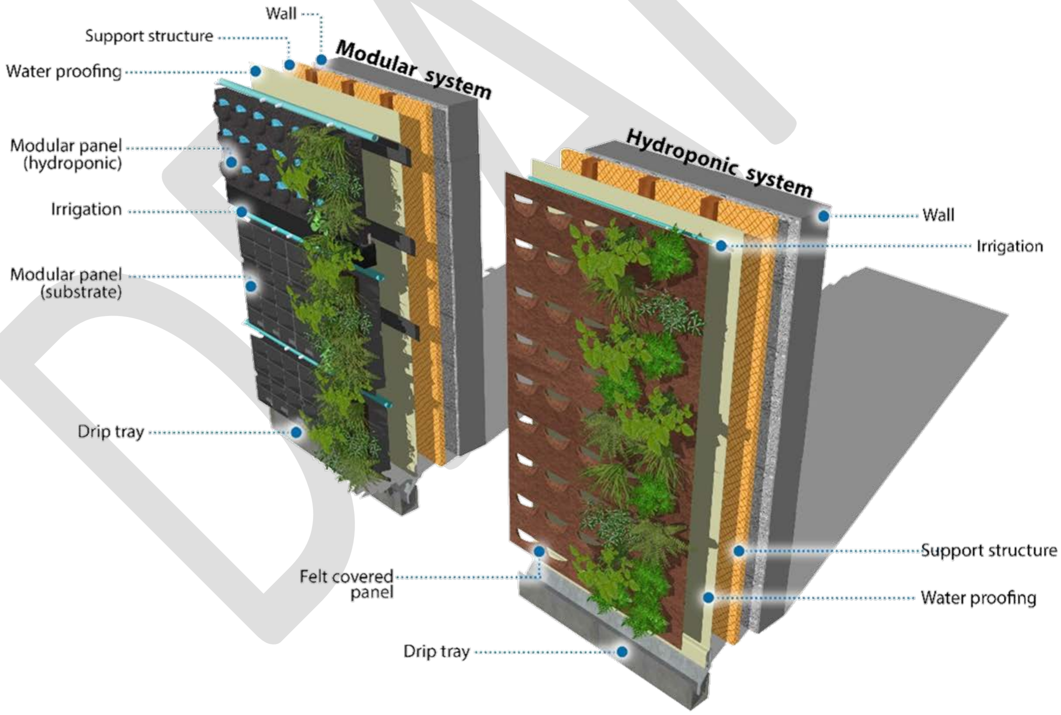

Figure 12-2. Types of living roof configurations (Adapted from State of Victoria, 2014, Growing Green Guide).

12.1.1 Major Components

The major components of living roofs and walls are:

Vegetation – vegetation is a critical component of living roof and wall practices. Vegetation takes up stored water from the soil media and releases it back to the atmosphere through transpiration. Everything from trees to low growing groundcovers can be used as vegetation based on the design of the practice. Plantings provide habitat for beneficial pollinators and birds, in addition to providing aesthetic benefits for the community.

Soil media – typically consists of a lightweight well-draining soil media that supports vegetation growth. The depth and components of the soil media varies by site conditions, application and vegetation selection. Many different proprietary mixes are available to meet a variety of project requirements.

Modular trays or panels – modular trays and panels are available as proprietary products in both roof and wall applications and are typically delivered to the site pre-vegetated for ease of installation and establishment.

Irrigation – although not required, irrigation systems are typically incorporated to ensure the health of the vegetation in times of extreme stress. They can be installed as drip irrigation or overhead spray systems depending on the requirements of the project.

Filter layer (roofs only) – woven fabrics or fleece that prevent the soil media from migrating into the drainage layer.

Drainage layer (roofs only) – aggregate, matting, or specially design panels that aid in the draining of the soil media. Drainage layers typically help to prevent standing water, but they may also be designed with depressions that create small reservoirs below the soil media layer for water retention.

Protection layer (roofs only) – also called a root barrier, these layers are placed directly above the waterproofing layer to prevent roots from compromising the integrity of the roofing material. A root barrier of at least 30 mils thickness is recommended.

Waterproofing layer or membrane – a watertight barrier must be provided to prevent rainwater from infiltrating the underlying structure. Watertight tar surfaces (conventionally used for roof sealing) are usually sufficient impermeable liners on roofs, but additional plastic or rubber membranes can be placed over the tar for added protection. The liner should be resistant to heat, desiccation, and ultraviolet radiation.

Support structure (walls only) – the rigid structural component that supports the weight of the vegetation and allows it to grow vertically. Support structures can either be free standing or mounted to the wall of a building.

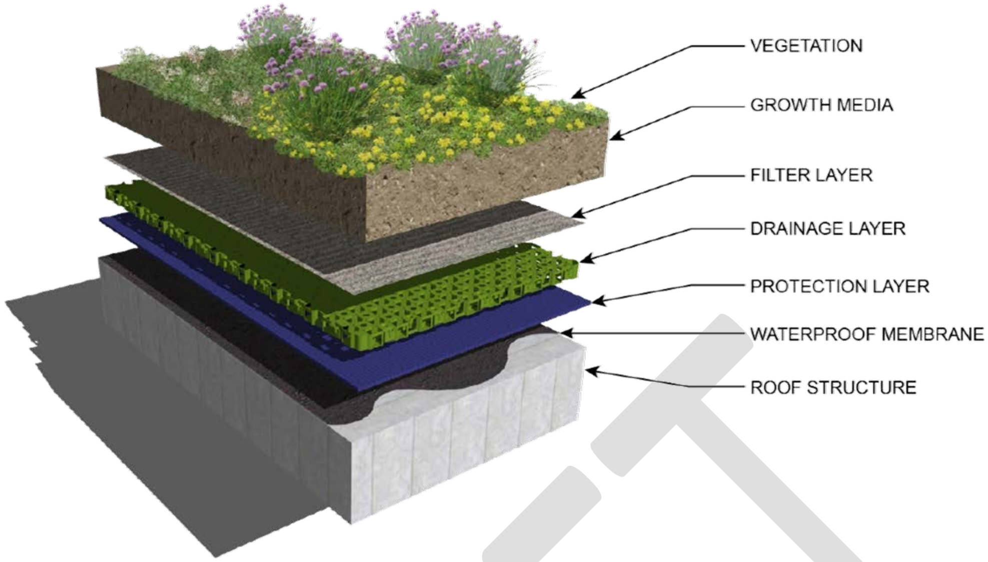

Figure 12-3. Single media assembly of a living roof

12.1.2 Configurations

Living roofs and walls come in a variety of configurations that can be customized to an individual site or building based on site factors and design goals. Living roofs are typically categorized into either extensive living roofs or intensive living roofs according to the selection of vegetation and depth of the soil media. Similarly, living walls can be applied in three general configurations based on where the soil media is located and if it is implemented on the wall of a building or on a retaining wall. Below are the basic configurations in which living walls and roofs are commonly applied.

Extensive Living Roofs

Living roofs with less than 6 inches of media and shallow-rooted vegetation are considered extensive. These roofs require little or no irrigation and contribute lighter loads to rooftops than intensive living roofs. Low growing plants form a dense carpet across the roof to protect the soil media from drying out and to crowd out weeds. Vegetation is typically composed of herbs, grasses, mosses, and drought tolerant succulents that can withstand extreme conditions and are capable of regenerating easily.

Figure 12-4 Extensive living roof

Intensive Living Roofs

When a living roof has more than 6 inches of media and features deeper-rooting plants, it is considered intensive. Intensive living roofs can be installed where structural support can handle the weight of deep, saturated soils and the live and dead loads associated


Figure 12-5 Green façade


Figure 12-6 Modular living wall

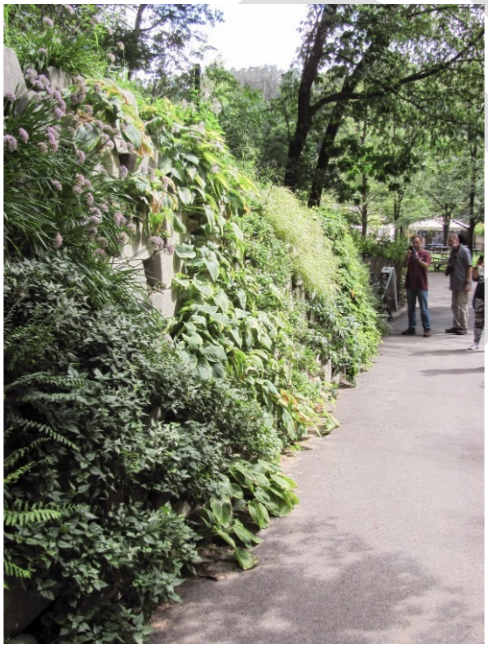
Figure 12-7 Landscape living wall

with taller vegetation. Often intended to function as small rooftop parks or gardens, intensive living roofs can provide many amenities; however, park-like landscaping on a rooftop typically requires more intensive maintenance and may require irrigation. Vegetation should be selected for drought hardiness and efficient water use, especially if limited rainwater harvesting options are available. Because of the wide variability in intensive living roof layout, media type and depth, irrigation demand and landscaping, the design process for intensive living roof systems is not covered in this manual. For more design guidance, contact a qualified professional with experience in implementing intensive living roofs.

Green Façades

Green façades, or green screens, incorporate climbing plants like vines to grow along support structures that are either attached to the building face or designed to be free standing structures to provide shade to selected areas. Soil for the plants is typically located at the base of the structure and several structures can be stacked vertically up the face of a building. Green façades typically take several years to mature.

Modular Living Walls

Modular living walls are typically constructed with pre-vegetated panels, blankets or bags that are attached to a structural support system on the wall of a building. The panels are typically constructed from plastic, synthetic fabrics, concrete, clay, or expanded polystyrene. Their increased structural support and available soil media throughout the practice allow for the use of a wider range of vegetation than what can be used in green façades. Everything from low growing groundcovers to small shrubs can successfully be planted in these practices. Vegetation can even be customized for full sun to shady conditions.

Landscape Living Walls

Engineered retaining walls that are designed for slope stabilization can also be vegetated on the face of the slope. Products used in living retaining walls are similar in function to the solid wall face options. They are often modular and come as interlocking units with additional space for planting soil. Overtime, the vegetation grown in living retaining walls will cover the structural supports and look like a solid wall of vegetation.

12.1.3 Site Suitability

Living roof and wall site applicability is largely dependent on structural constraints of buildings in relation to stormwater management goals and other project goals. Considerations for these practices include careful characterization of building substrates (roof slope, roofing materials, wall materials, structural capacity, etc.) and cost of implementation versus other GSI options (NYC DEP, 2012). Flat or low-slope roofs found in many urban and commercial settings may provide a viable volume reduction option, especially where ground level options are limited or costly.

12.2 Design Process

Designing living roofs and walls requires careful consideration of any structural requirements and existing site conditions to ensure the proper function of the practice. This section provides an overview of the design process with detailed discussions of design and material selection for each component in the following sections. Additionally, Table 12-1 lists chapters that are frequently cross-referenced for any applicable design methods and requirements.

Design requirements and recommendations are listed only for applicable for materials that are not part of proprietary living roof or wall systems. Typically, living walls are constructed from pre-manufactured systems that are designed by the manufacturer. Many proprietary living roof systems exist as well. Under these circumstances, design of the system should be per manufacturer's specifications, if all the appropriate structural considerations have been properly addressed.

Table 12-1 Cross-referenced chapters

Chapter 2	Regulatory requirements
Chapter 3	Site assessment Conceptual design
Chapter 4	Runoff volume calculations
Chapter 6	Properties of soils and aggregates Soil water calculations
Chapter 8	Vegetation selection

12.2.1 Site Investigation

The ability to retain stormwater on site and incorporate living roof and wall practices into the site's stormwater management approach depends on many factors that must be evaluated. A more detailed explanation on how to conduct a thorough assessment can be found in Chapter 3, Site Design and Stormwater Management. Because of the complexities and risks of locating stormwater infrastructure on a building roof or wall, the designer should not approach site evaluation and practice design without consulting structural engineers and horticultural professionals with living roof or wall design experience.

As part of the overall site assessment, the following attributes specifically apply to living roof and wall practices and shall be addressed:

- Utility conflicts including mechanical equipment associated with the building
- Structural integrity of existing building materials
- Reflected light from nearby surfaces
- Sources of exhaust, or warm and cold air emissions
- Intensity of solar radiation
- Areas of precipitation deflection
- Climate and microclimate factors

12.2.2 Living Roof Design Considerations

When designing living roofs, there are a few common considerations that need to be addressed. The section below details the most pertinent of these considerations.

Figure 12-8 Sloped living roof installation in Alaska

Roof Pitch

Living roofs can be installed not only on conventional flat roofs but also on roofs with slopes up to 45^{o} (100%). As roof pitch increases, the rate of runoff from the roof increases as well. Additional design considerations for water retention, shear and slide are needed on roofs having slopes of 5^{o} (8.8%) or more. Furthermore, it is recommended that the roof have a slope of at least 1.1^{o} (2%) to prevent ponding in the vegetation substrate. On roofs having less than a 2% slope, special roof dewatering measures must be designed into the practice (FLL, 2008).

Structural Capacity of Roof

Living roof suitability and design depend on the excess load that can be applied to a rooftop. **A qualified structural engineer should be consulted to determine the structural capacity of the roof in question to support additional dead and live loads resulting from living roof installation**. For new construction, the building designer might consider the additional roof load in selecting building structural components. In either scenario, the dead and live roof loads from the living roof installation will depend on the specific living roof components and must be evaluated case by case. The procedure to evaluate allowable loads for living roof projects is outlined in ASTM E2397 Standard Practice for Determination of Dead Loads and Live Loads Associated with Vegetated (Green) Roof Systems (ASTM, 2015).

Wind Loads

Chapter 3, Site Design and Stormwater Management details many of the climate and microclimate factors that affect the design of living roofs. However, wind creates specific challenges for living roofs. The surrounding terrain, height of the building, design of the parapet, and direction of prevailing winds all must be accounted for during design. Wind pressure, both positive and negative, can damage living roofs. Positive pressure can scour, or dislodge soil media and uproot vegetation, whereas negative pressures can cause materials to be lifted off the roof. As wind passes over a roof edge or parapet, winds begin to swirl and can scour vegetation and soil media as well (Figure 12-9). All living roofs shall be designed in accordance with recommendations in "Wind Design Standard for Vegetative Roofing Systems" (ANSI/SPRI RP-14 2016).

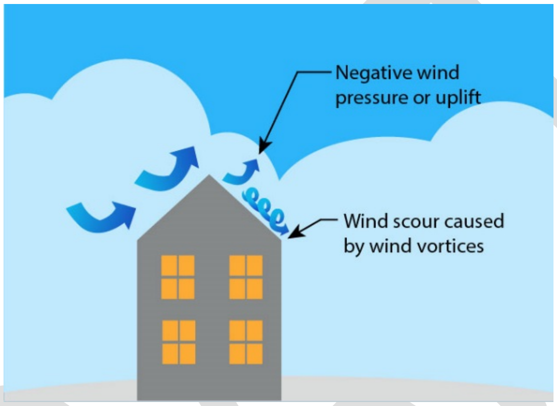

Figure 12-9 Effects of wind on roofs

Furthermore, as buildings increase in height, the effects of wind and temperature become more pronounced. There is little research regarding the height at which living roofs should not be used. All living roofs placed at heights greater than 20 stories shall be designed by professional engineers and living roof designers with experience designing systems for tall buildings. Practices at this height require the installation of an irrigation system.

Fire

Sparks and radiated heat can cause the organic components of living roofs to catch fire. It is recommended that all living roofs demonstrate a resistance to fire and flame spread. The following design recommendations can help to increase resistance (FLL, 2008):

- Soil media with low organics and a depth of no less than 2 inches
- Provide stone border or concrete paver blocks in a minimum width of 20 inches between vegetation and any openings in the roof, such as skylights or roof vents, or under any openings in vertical walls, such as doors or windows
- Provide a vertical barrier (minimum of 12 inches high) or 40-inch wide strip of stone or concrete pavers every 150 linear feet of practice

12.2.3 Living Wall Design Considerations

Structural Considerations

Selection of the living wall system is dependent upon if the building can support the necessary vertical gravity loads and lateral wind loads. A structural engineer should be consulted in making these decisions.

Irrigation water availability

Nearly all living wall applications will require irrigation for the vegetation. In these instances, a water source shall be identified during design for this purpose.

12.3 Living Roof Design Elements

The following section provides greater detail of elements that are common to all living roof practices.

12.3.1 Drainage Layer

A drainage layer, also known as a drainage net or sheet drain, is necessary to convey excess rainwater to the roof drains. This layer also maintains an aerobic root zone for plant health. The drainage layer should be designed to convey the 10-year runoff event without backing water up into the growing media. A geotextile should be placed between the media and the drainage layer to prevent migration of media and act as a root barrier.

Figure 12-10 Living roof drainage layer

Figure 12-11 Typical composition of lightweight soil media for living roofs

12.3.2 Soil Media

Living roofs can be designed as flow-through systems or can be designed to detain a specific design volume of water (as determined by a qualified structural engineer). The sizing methodology presented in Chapter 8, Bioretention, can be used to design the system to capture a specific design volume.

Soil media for living roofs should have the following characteristics:

- Well drained and aerated
- High porosity
- High nutrient holding capacity (cation exchange capacity)
- Permanent (non-biodegrading)
- Lightweight
- Windproof
- Stable (must be able to physically support plants)

Several media types are available from living roof component suppliers, but generally expanded lightweight aggregates are preferred (e.g., expanded slate, expanded shale, expanded clay). For extensive living roofs, between 4 and 6 inches of media should be provided. The specifications provided in Table 12-2 are example parameters that should be specified as part of the design and included in the construction documents. Intensive living roofs should also employ lightweight aggregate media, but structural capacity generally allows a wider range of soil materials. Living roof media installation can be challenging and may require the use of a crane, auger, conveyor, or pneumatic delivery system.

Table 12-2. Example living roof media specifications (Source: Dorman, et al, 2013)

Parameter	Specification
Non-capillary pore space at field capacity	15% (vol)
Moisture content at field capacity	12% (vol)
Maximum media water retention	30% (vol)
Alkalinity, Ca CO_3 equivalents	2.5%
Total organic matter by wet combustion	3-15% (dry wt.)
pH	6.5-8.0
Soluble salts	6 mmhos/cm
Cation exchange capacity	10 meq/100g
Saturated hydraulic conductivity for single media assemblies	0.05 in/min
Clay fraction (2 micron)	0
Pct. passing US#200 sieve (i.e., silt fraction)	5%
Pct. passing US#60 sieve	10%
Pct. passing US#18 sieve	5%-50%
Pct. passing 1/8-inch sieve	20%-70%
Pct. passing 3/8-inch sieve	75%-100%

12.3.3 Outlet Design

As with all roofs, components must be incorporated into the roof structure to allow free drainage of excess runoff from the rooftop and away from the building. For extensive living roof applications, drainage components can include internal roof drains or roof scuppers along roof perimeters. These components should be designed in accordance with local building codes. To ensure adequate conveyance of roof runoff from the drainage layer to the outlets, living roofs should be set back a minimum of 12 inches from roof drains. The area surrounding the roof drains should be filled with washed MDOT 6A or 6AA stone, or alternative high-porosity material. Placing a light-colored stone buffer around the roof drains also delineates a no-plant zone for maintenance staff (Figure 12-12). The no-plant zone should remain free of vegetation to prevent drain clogging.

Figure 12-12. Light-colored gravel delineates the no-planting zone for maintenance personnel

12.3.4 Edging

Edging is used in any area were the living roof vertically abuts another material or if there is not a structural support at the edge to prevent soil media or any other loose materials used in the practice from washing out. Edging shall be sturdy and not generate pressure on waterproof linings or root penetration barriers. Edging is typically constructed from aluminum, lightweight concrete, fiber cement, or plastic.

Figure 12-13 Aluminum edging

12.4 Living Wall Design Elements

12.4.1 Wall Planters

Modular living walls are typically constructed from a series of individual planters made from felt, aluminum or plastic. These planters often come pre-planted and are installed on frames and rails that come with the system.

Figure 12-14 Wall mounted planters

12.4.2 Green Façade Support Structures

There are two primary types of support structures used in green façades. They include modular trellis systems and cable systems. Modular trellis systems consist of rigid lightweight panels that are either mounted to a wall or used as free-standing systems. Cable systems use anchors and spacers attached directly to building walls or frames to secure high-tensile steel cables in a variety of patterns.

12.5 Vegetation

Selection of vegetation for living roofs and walls is highly variable based on specific site conditions. Everything from trees on intensive vegetated roofs to low growing alpine plants on modular living walls can be used. All vegetation selection is highly dependent on the height at which it is installed, the amount of sunlight it receives, if irrigation is being used, how much maintenance will be provided, and the depth or presence of soil media. It is highly recommended that living wall and roof manufacturers or horticulturalists specializing in these applications be consulted to ensure the appropriate selection of vegetation for the practice.

Figure 12-15 Green façade wall-mounted trellis system

Figure 12-16 Typical low-growing living roof vegetation

Figure 12-17 Pre-vegetated trays

Extensive Roofs

Vegetation for these practices should consist of low-growing, extremely drought-tolerant, biodiverse species adapted to survive in the harsh environment of a rooftop. Succulents, like sedums and sempervivums store water in their fleshy leaves and are highly suited for this environment. Appropriate vegetation should be selected based on the specific site conditions and recommendations by local horticulturalists and living roofs manufacturers. Additionally, pre-vegetated trays (Figure 12-18) may be purchased to significantly reduce the amount of time it takes for the plants to establish.

Figure 12-18 Young twinging vine on a trellis structure

Living Façades

The most common type of plant used in living façade applications is vines. Different vines have characteristics that may make them more well suited than others for living façade applications. Twining vines, like honeysuckle, wind around vertical and horizontal supports, whereas tendril vines like sweet peas have tendrils that extend from the stem of the plant to wind around supports as the plant grows. These types of vines do not require any additional effort to encourage upward and outward growth. Running vines like tomatoes, are not able to attach to supports themselves, and will need to be woven through the support structure or tied to it. Clinging vines like English ivy attach to surfaces with aerial rootlets. When these plants are specified, it may be necessary to create a separation between the structural support of the living wall and the building face, as ivies can damage brick or wood surfaces.

Modular Living Walls

Most modular living walls can accommodate a wide variety of plants that are well suited to the conditions that will present in the location of the wall. Plants can be selected based on sun/shade requirements, and can also include annuals that will need to be replanted each year. The most well-adapted plants for modular living wall applications are often found in alpine environments where they grow at high elevations above the tree line. These environments experience extreme temperatures, drying winds, and significant amounts of ultraviolet radiation, which are like what is faced in the harshest of living wall conditions.

Figure 12-19 Shade loving annuals planted in a modular living wall

Landscape Living Walls

Plants used in landscape living walls are typically drought resistant plants that are well suited for landscape applications that have similar sun/shade requirements. Figure 12-21 shows hostas, Japanese forest grass, coral bells and other perennials that are well suited for shady sites. Landscape living walls planted in full sun are typically planted with succulents and grasses.

Figure 12-20 Shade loving plants growing on a landscape living wall

12.6 Irrigation

The need for irrigation depends on the plants selected, microclimate of the practice and roof slope, where applicable. Most living wall practices require irrigation given the small volumes of soil used or the lack of soil all together. Typically, irrigation is not required on roofs when properly designed but it is recommended as a backup during times of severe drought or prolonged wind, heat or sun exposure. Irrigation should be achieved using air conditioner condensate or harvested rainwater whenever possible. If sufficient water is not available from these sources, either deeper media with higher water holding capacity can be specified, or a water source may need to be located on the roof.

Drip Irrigation vs. Overhead Spray

In typical landscaping applications, subsurface drip irrigation is much more efficient when it comes to irrigating plants without waste (Camp, 1998). This is especially true when it comes to irrigating individual plants in the landscape. However, due to the use of predominantly inorganic soils on green roofs, there is limited capillary movement of water through the soil column, which limits upward and lateral movement of water. According to a Michigan State University study, overhead irrigation is much more efficient than drip irrigation in living roof applications, provides more even water

Figure 12-21 Overhead spray irrigation on vegetated roof

When is irrigation recommended:

- In all living wall applications
- Roof pitch steeper than 15%
- Walls with southern exposure
- Roofs or walls exposed to strong winds (high roofs, wind corridors)
- Precipitation is limited (under canopy or roof overhang)
- Selected vegetation and soil depths require additional moisture

distribution, better water retention, and increased plant health (Rowe, Kolp, Greer, & Getter, 2014).

The choice between drip irrigation and overhead spray applications will depend on the nature of the project, however. Some proprietary systems come with subsurface or drip irrigation included, and these systems may be necessary in applications where high winds are common. If drip irrigation is used, including a moisture retention fabric has been shown to improve irrigation efficiencies (Rowe, Kolp, Greer, & Getter, 2014).

12.7 Layout

The location and sizing of living roof and wall practices needs to account for a variety of factors during the design process to ensure there is sufficient space to incorporate the practice with other building requirements and that the practice is accessible for routine maintenance activities. The section below details the most common layout considerations.

12.7.1 Maintenance Access

Figure 12-22 Concrete paver paths for maintenance access

Regular maintenance is a key component to ensuring the long-term functionality of living roof and wall practices. Typical maintenance activities such as weeding, irrigating and inspection of structures, requires considerations for access during the design. It is recommended that gravel or paver paths be incorporated into the design for maintenance purposes to prevent staff from walking on the plants. A full list of maintenance activities is presented in Section 12.10. It is recommended that this section be reviewed to ensure that proper access is provided for their execution.

12.7.2 Pedestrian Spaces on Roofs

Figure 12-23 Pedestrian space on living roof

Living roofs can be designed around specific pedestrian areas that incorporate specialized pavers and decking materials for use on roofs. These areas should be identified and designed along with the design of the living roof to ensure adequate space is provided for each use.

12.7.3 Sizing the Practice

Once a suitable location has been determined and the appropriate configuration has been selected, the area of the practice can be determined. Determining the area is an iterative process that frequently requires further modifications to many design elements before a suitable design solution can be found. The following sections and chapters within the manual should be referenced for additional sizing criteria:

- Section 12.8 below includes design standards and requirements for each infiltration practice configuration.
- Infiltration calculation methods are provided in Chapter 6, Soil, Water and Aggregates.
- Chapter 8, Bioretention details additional considerations for selection of plant material

12.8 Design Standards

Design standards and requirements are provided in this section. General requirements which apply to both living roofs and living walls are presented first. Following the general requirements section, standards that are unique to each type of practice are presented. The design and construction of living roof and wall practices shall meet all general requirements, as well as the practice specific requirements.

12.8.1 General Requirements

This section presents general requirements that must be met for all types of living roof and wall practices.

- All structural loading requirements shall be verified by a licensed structural engineer prior to installation of the practice.
- All manufacturer recommendations shall be adhered to for the installation of their products.
- Permanent access for maintenance is required. This includes physical access to the roof for living roofs and vehicular access with a minimum nine-foot width for living walls.
- A planting plan is required for all practices. The planting plan shall include the following:
    - Plan view (roof) or elevation view (wall) of the practice
    - Vegetation selection, plant spacing, and installed sizes

12.8.2 Living Roofs

The following section details additional requirements that are specific to the design of living roofs. The contents of this section do not preclude the necessity to meet any of the other requirements applicable to living roofs contained within this chapter or within this manual.

- Roof pitch shall not exceed 45° (100%) in areas where living roofs are installed.
- Provisions shall be provided to prevent falls during construction and routine maintenance of the living roof. This includes temporary railings or roof scaffolding during construction.

- All materials used for the roof and living roof shall be selected such that they do not negatively affect the growth of plants. This includes any chemical incompatibilities which may result in the off gassing of noxious chemicals.
- Roof drainage within the vegetated area shall contain a minimum of at least one runoff facility and one emergency overflow.
- Roofing materials and roofing joints that are not resistant to moisture or root penetration shall be sealed with either membranes or liquid sealant prior to proceeding with installation. This includes any areas where fixtures or fittings protrude through the roof.
- All waterproofing on roof with slopes greater than 3^o (5.2%) shall be fixed in place with an appropriate adhesive.
- All waterproofing and root resistant membrane materials shall be certified for use in applications with constant exposure to water and resistance to hydrolysis.
- Living roofs shall provide verification that they are resistant to sparks and radiated heat.
- Roofs with slopes greater than 15^o (26.8%) shall provide structural anti-shear and slip protection such as special matting, anti-shear plates or fabrics, studded plates or other devices. Roofs with slopes greater than 30^o (57.7%) require calculations to be completed by a professional engineer with prior experience designing living roofs on steep slopes.

12.9 Construction Considerations

12.9.1 Materials Storage and Delivery

During construction, living roof materials must be transported to the rooftop. This can be done via ladder lifts, elevators, or human physical labor. The most efficient method is typically using a crane. Media can be pneumatically blown onto the roof surface. Adequate areas must be available at the building perimeter for material and equipment staging.

12.9.2 Supplemental Irrigation during Plant Establishment

In the plant establishment phase, supplemental irrigation may be necessary to ensure plant survival and full vegetation coverage of planting media and/or support materials. In these instances, water may need to be transported to the site if it is not readily available.

12.10 Operation and Maintenance

Inspection and maintenance are critical to ensuring safe and effective functioning of living roofs. Table 12-3 provides specific inspection and maintenance tasks.

Table 12-3. Inspection and maintenance activities for living roofs

Task	Frequency	Indicator maintenance is needed	Maintenance notes
Media inspection	2 times/year	Internal erosion of media from runoff or wind scour, exposed underlayment components	Replace eroded media and vegetation. Adopt additional erosion prevention practices as appropriate.
Liner inspection	1 time/year	Liner is exposed or tenants have experienced leaks	Evaluate liner for cause of leaks. Repair or replace as necessary.
Outlet inspection	2 times/year	Accumulation of litter and debris around the roof drain or scupper or standing water in adjacent areas	Litter, leaves, and debris should be removed to reduce the risk of outlet clogging. If sediment has accumulated in the gravel drain buffers, remove and replace the gravel.
Vegetation inspection	1 time/year	Dead plants or excessive open areas on living roof	Within the first year, 10% of plants can die. Survival rates increase with time.
Invasive vegetation	2 times/year	Presence of unwanted or undesirable species	Remove undesired vegetation. Evaluate living roof for signs of excessive water retention.
Temporary watering	1 time/2-3 days for first 1-2 months	Vegetation has not yet reached maturity after one growing season, or if vegetation begins to wilt during extended periods of drought/heat	Watering after the first year might be required.
Winterize irrigation system	1 time/year	Nighttime temperatures are approaching freezing in Autumn.	Freezing temperatures can cause pipes to burst and can damage the irrigation system if one is used.

12.11 Design Checklist

The design of living roofs and walls typically involves a few iterations, design of several individual components, and frequent modifications during the design process. To ensure that the practice has been properly designed, the following checklist shall be used upon completion of the design, but before construction drawings have been finalized. This checklist can then be included as part of the Post-Construction Stormwater Management Plan along with any required calculations to document the design.

Include design checklist here

12.12 References

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13. Stormwater Wetlands

Stormwater wetlands include shallow pools that store stormwater between events and promote the growth of rooted vegetation such as reeds, rushes, willows and cattails. Stormwater wetlands are one of the most effective GSI techniques at removing pollutants from the stormwater runoff. They perform like large detention practices for temporarily storing water and releasing it slowly. Stormwater wetlands only reduce the runoff volume through evapotranspiration and therefore are limited for volume reduction strategies. This chapter discusses basic information on locating and siting a stormwater wetland along with standards for design.

Contents

13.1 Introduction

Stormwater wetlands are engineered, shallow-water ecosystems designed to treat stormwater runoff. Commonly implemented in low-lying areas, stormwater wetlands are well suited to areas along river corridors where water tables are higher. Sediment and nutrients are efficiently reduced by stormwater wetlands by means of sedimentation, chemical and biological conversions, and uptake. Stormwater wetlands provide flood control benefits by storing water and slowly releasing it over 2 to 5 days. In addition to stormwater management, stormwater wetlands provide excellent plant and wildlife habitat and can often be designed as public amenities. Research has indicated that a home located next to stormwater wetlands can have a 20 to 30 percent higher selling price (Russell, Teague, Alvarez, Dantin, & Nestlerode, 2012).

Figure 13-1 Example Surface Flow Stormwater Wetland

13.2 Practice Selection and Design

Stormwater wetlands offer aesthetic, wildlife habitat and water quality advantages over extended wet detention practices (Chapter 7, Detention Practices). Wetlands, however, require more space for the same volume being managed. The design and construction of stormwater wetlands are constrained in a fashion like an extended wet detention practice. Stormwater wetlands are commonly shallow and more difficult to enter, hence safety issues may be reduced compared to large stormwater management practices (WEF and ASCE, 2012).

Runoff passes through a sediment forebay before entering the stormwater wetland. The forebay provides an opportunity for sediment to settle and for debris to be captured. Runoff fills the wetland design volume to a depth of 12 inches or less and slowly drains over 2 to 5 days through a drawdown orifice installed at the elevation of the permanent pool. Runoff from large storm events can either be diverted (offline design) to the downstream stormwater network or can be detained (online design) using a riser structure or weir. The discharged peak flow rates may be managed like a detention practice (Chapter 7) by providing storage and an outlet control structure. Reducing the volume of runoff is limited evapotranspiration. Infiltration through the bottom of the wetland is discouraged to ensure that permanent pools are maintained for plant survival and aesthetic purposes.

13.2.1 Siting

In general, stormwater wetlands are particularly well suited to low-lying sites with large drainage areas. The configuration of the stormwater wetland will vary by site and can be adapted to the available space and desired functions. Long, linear wetlands can be installed along the perimeter of sites, smaller pocket wetlands can be distributed throughout a development, or larger wetlands can be installed at the downstream end of a catchment.

Constructed stormwater wetlands are typically constructed in the lowest area of a site such that runoff can be conveyed by gravity flow and so that excavation is minimized. The stormwater wetland location should provide adequate elevation difference, typically 3 feet or more, to discharge water by gravity flow. Constructed wetlands can be incorporated along the perimeter of a site by designing a long, linear footprint, or it can serve as an attractive amenity in shared areas of developments. If the entire design volume cannot be stored in one location or if utility conflicts are apparent, wetland pockets can be distributed between several locations and connected with vegetated channels or buried conduit.

13.2.2 Typical Configuration

Stormwater wetlands are often irregularly shaped and include: an inlet, forebay, storage pools, outlet, and an emergency overflow. The wetlands are often constructed with organic configurations including embayments, islands, and peninsulas to provide diverse habitat and improve the functionality of the wetland. Figure 13-2 illustrates an example configuration.

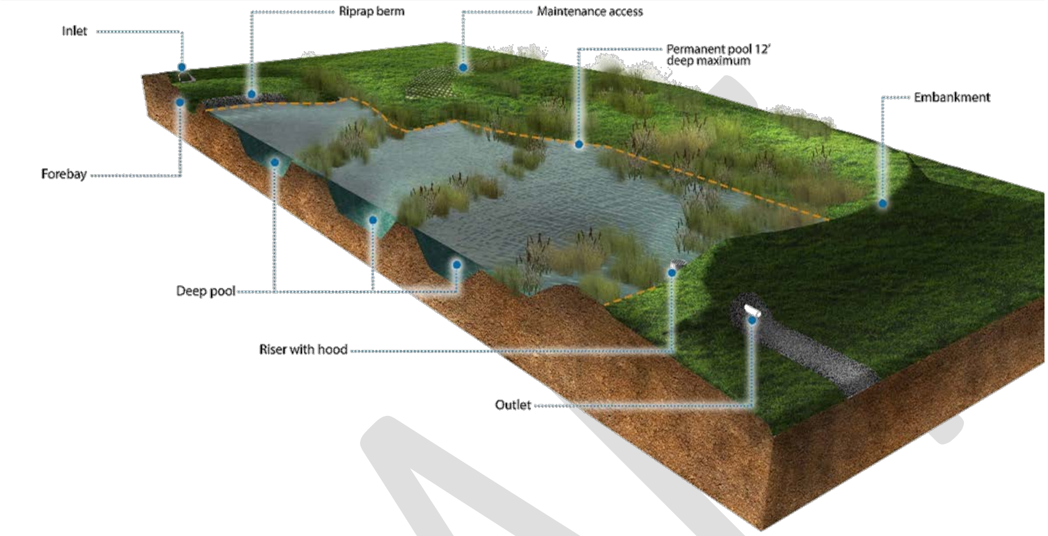

Figure 13-2 Example Stormwater Wetland Configuration

13.2.3 Soils

If possible, organic soils (muck) from displaced wetlands should be stockpiled and used in the constructed stormwater wetland. Do not use soils from wetlands that have nuisance or invasive species. Topsoil or peat may be substituted for wetland muck.

A 6- to 12-inch layer of wetland muck, topsoil or peat should be used. Depth of soil will depend on specified plantings and underlying soil characteristics—consult a plant specialist as needed. Native soils excavated in construction can be used, but a soil test should confirm that the soils contain adequate nutrients for plant survivability (subsoils tend to be relatively infertile, so topsoil should be separately stockpiled for this purpose). Soils should not contain excessive levels of phosphorus (greater than 15 ppm) because this nutrient tends to dissociate from the soil under saturated conditions and may be exported out of the system.

13.2.4 Components

Stormwater wetlands are constructed with several different components depending on site conditions and the desired results. The inlet, forebay, outlet, and emergency overflow are essentially the same as large detention practices. For basic information refer to Chapter 7, Detention Practices. Additional characteristics unique to stormwater wetlands are further discussed next.

Figure 13-3 Stormwater Wetland Inlet

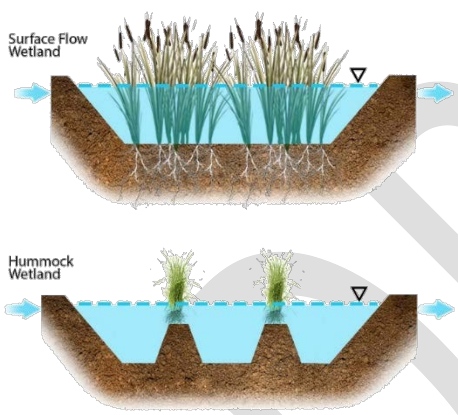

Figure 13-4 Wetland Variants

Inlet

Stormwater wetlands may be constructed either as on- or offline configurations. An offline design is preferred to limit the inundation during very large storm events. Offline configurations provide for a high flow bypass at, or immediately upstream of, the inlet.

Forebay

Forebays are separated from the downstream wetlands. A means of draining the forebay while maintaining water in the wetland should be provided to accommodate maintenance activities.

Permanent Pool

Sizing the permanent pool is the same as an extended wet detention practice. An additional sizing criterion is to minimize the risk of drying out the wetland during droughts. To prevent the permanent pool from drying out the volume should be at least 2 times the volume of evapotranspiration for the duration of the drought at summer evaporation rates. Refer to Chapter 4, Hydrologic Procedures, for additional information on evaporation.

Constructed stormwater wetlands may be configured as *surface flow* or *hummock* wetlands. A surface flow wetland is an open water system (depths less than 2 ft.) and completely covered with rooted vegetation. The cross-section of a hummock wetland alternates between shallow pools with vegetation and deep pools (greater than 3 ft.) devoid of rooted vegetation. Hummock wetlands tend to be smaller than surface flow wetlands. Field data have not established significant differences in the performance of these wetland configurations although hummock wetlands are reported to slightly improve nitrification and may reduce mosquito populations (Thullen, Sartoris, & Walton, 2002). To prevent short circuiting, the berms in a hummock wetland are placed laterally across the wetland. (WEF and ASCE, 2012)

A manually operated valve should be provided at the lowest possible stage of the wetland to allow drawdown for maintenance. The intake should be protected with gravel or a trash rack, or both, to minimize clogging and be sized one standard pipe size larger than would be needed to dewater the entire wetland within 24 hours. The valve should have locking features to prevent unauthorized dewatering.

Outlet

The outlet allow water to exit the storage pool. There are countless outlet configurations that may be used. The most common outlet includes a riser pipe with a series of orifices to allow the slow release of water, and an overflow grate on top of the riser pipe for the passage of larger events. Weir structures may be more appropriate as a low flow outlet based on the limited hydraulic heads. There is not a standardized procedure to find the optimal outlet combination. Many different combinations among orifices, culverts, and weirs achieve multiple-event flow control and water quality benefits. Other appurtenances such as trash racks, backflow preventers, odor control systems, and valves may be included to provide a variety of benefits.

Emergency Overflow

An emergency overflow or spillway allows excess water to pass during floods that exceed the practice's design specifications. Emergency overflow systems are commonly a low point in the side of the practice. Care must be taken to direct overflow water to areas that will not cause problems or damage to nearby infrastructure.

Maintenance Access

To maintain stormwater wetlands, maintenance crews and equipment must occasionally access wetland components. Wetland design should incorporate a dedicated access easement from a public road to the wetland and an appropriate maintenance staging area. The grading plan for the wetland should incorporate access paths as appropriate for maintenance equipment to reach critical maintenance points including, for example, the forebay and outlet. The site geotechnical analysis will determine whether the access path must be stabilized to support heavy equipment.

13.2.5 Other Design Issues

Plant Selection and Establishment

Although wetlands are typically wet, most native wetland plants are well adapted to surviving extended periods of drought. Emergent plant survival rates, however, dramatically decrease when normal water depth exceeds 6 inches and invasive plants can begin to establish monocultures. Monocultures of reeds and cattails tend to provide refuge for mosquitos. For these reasons, it is critical that a diverse selection of flowering, emergent vegetation is planted in a maximum of 3 to 6 inches of water. This will provide the optimum habitat for mosquito predators, such as dragonflies, and reduce plant die-off. At least three species, preferably more, should be planted in each zone of the wetland.

Figure 13-5 Tollgate Stormwater Wetland (Lansing Township, MI)

Although trees and shrubs can provide habitat, shade, and aesthetic benefits, take care to immediately remove woody vegetation from embankments to prevent geotechnical failures.

Design for Multi-Use Benefits

Stormwater wetlands can provide excellent ecosystem services and aesthetic value. In addition to enhancing biodiversity and beautifying the urban environment with native vegetation, the following components can be incorporated into stormwater wetlands to promote multi-use benefits:

- Simple signage or information kiosks can educate the public on the benefits of watershed protection measures or provide a guide for native plant and wildlife identification.
- Boardwalks, wildlife viewing platforms, and benches can be provided to encourage interaction.
- Volunteer groups can be organized to perform basic maintenance as an opportunity to raise public awareness.
- Wetlands can be used as outdoor classrooms for school science projects and field trips.

Figure 13-7 Channel connecting wetlands zones

Figure 13-6 Neighborhood stormwater wetland with multi-use pathway

13.3 Design Standards

Design standards and requirements are provided in this section.

13.3.1 Placement on the Site

The following criteria shall be used to define the layout and placement of constructed stormwater wetlands.

- Stormwater wetlands should not be constructed on steep slopes, nor should slopes be significantly altered or modified to reduce the steepness of the existing slope to install a wetland.
- Stormwater wetlands should not be constructed within 10 feet of the property line.
- Stormwater wetlands should not be constructed within 10 feet of the City's sanitary or combined sewer.
- In areas with high quality soils or well-draining soils, GSI practices which emphasize infiltration should be constructed instead of a stormwater wetland. If a stormwater wetland practice is selected despite the presence of well-draining soils, the bottom must be lined to prevent infiltration.
- Stormwater wetlands must have sufficient easements for maintenance purposes. Easements should be sized and located to accommodate access and operation of equipment, spoils, deposition and other activities identified in the development's stormwater management plan.

13.3.2 Hydrologic Requirements

Refer to Chapter 2, Regulatory Requirements, for site specific design requirements. The hydrologic requirements specified here apply to all constructed stormwater wetlands.

- The practice shall be designed to meet the applicable site-specific flow requirements as discussed in Chapter 2, Regulatory Requirements.
- If designed as an online system, the practice shall be designed to safely pass a 100-year storm. If the 100-year storm is not specified as part of the site-specific design requirements an emergency outlet or spillway capable of conveying the 100-year design storm must be included in the design.
- The design shall prevent erosion throughout the entire wetland including but not limited to the inlet(s), forebay, outlet, emergency overflow, side slopes and embankments. Erosion may be controlled with hard armoring techniques or vegetation.

13.3.3 Layout and Geometry

Requirements

- A minimum of one (1) foot of freeboard is required above the design water level of a stormwater wetland practice except at the emergency overflow location.
- Stormwater wetlands shall be designed with safety considerations including reducing the chance of drowning using warning signs, reducing the maximum depth, or including benching and mild slopes, or any combination thereof, to prevent people from falling in and to facilitate their escape from the wetland.
- Side slopes shall not be steeper than 3H:1V. Terraced slide slopes are allowed however the maximum vertical rise is limited to 18 inches. Refer to Figure 13-8.

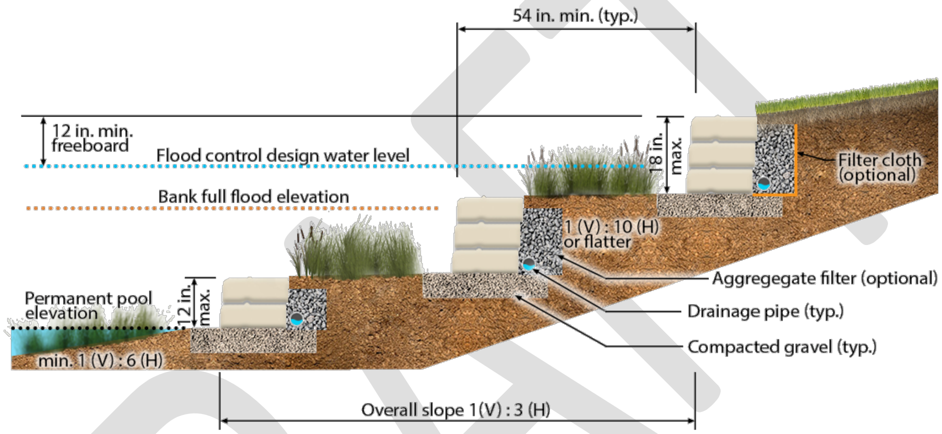

Figure 13-8 Terraced Side Slopes

- Design outlet structures to minimize risk of a person being pushed, pinned or sucked into outlet pipe.
- The inlet and outlet should be as far apart from one another as practical. Avoid situations where the inlet and outlet pipes are directly across from each other and only a short distance apart.
- Permanent access must be provided to the forebay and outlet. It shall be at least nine feet wide, have a maximum slope of 15 percent, and be stabilized for vehicles.

Preferred Design Elements

- Additional considerations for side slopes includes:
    - 6H:1V is the preferred maximum side slope (ASCE, 2014).
    - 10H:1V is the recommended side slope where space is available.
    - 20H:1V is the minimum recommended side slope.
- Irregularly shaped wetlands are acceptable and encouraged to improve site aesthetics.
- Trash/safety racks should be considered on a case-by-case basis. Hinged racks facilitate cleaning.

13.3.4 Pretreatment

A forebay or other pretreatment system is highly recommended at all major inflow points to capture coarse sediment, prevent excessive sediment accumulation in the main pool, and minimize erosion by inflow. Stormwater runoff that has already passed through another GSI practice does not need to pass through a second pretreatment device.

Preferred design elements for a forebay include:

- Size the volume of the forebay to contain 10 percent of the water quality treatment volume.
- Forebays shall have a minimum length of 10 feet.
- Forebays shall have a depth of 3 to 6 feet near the inlet, then sloping up to 2 to 3 feet deep toward the berm (incline dissipates energy and promotes particle setting). Increased depths can be provided for sediment storage.
- The forebay shall be capable of being drained while water persists in the wetlands.
- The bottom of the forebay shall be hardened (for example with concrete or grouted riprap) to make sediment removal easier.
- Physically separate the forebay from the primary storage pool with a berm, gabion wall, or other divider.
- Flows exiting the forebay must be non-erosive.
- Install a permanent vertical marker that indicates the sediment accumulation depth.

The forebay storage volume counts toward the overall storage volume required if the forebay is dewatered between rain events.

13.3.5 Permanent Pool

Requirements

Volume

- The permanent pool shall be at least the water quality treatment volume for the drainage area.
- The permanent pool shall be more than 2 times the volume of evapotranspiration during drought conditions at summer evaporation rates (Chapter 4, Hydrologic Procedures).

Surface Area

- The area of the open surface water areas (i.e. the forebay, outlet, and open surface water areas in the wetland) shall be 30 to 50% of the total area (forebay, outlet and all the wetland area).
- The remaining 50 to 70% shall be wetland emergent zones.

Wetland Zone with Emergent Vegetation

- Wetland zones with emergent vegetation shall have depths of 6 to 18 in. One-third to one-half of the wetland zones with emergent vegetation shall be approximately 6 in. deep. Water depths are based on dry weather conditions.
- A wetland zone with emergent vegetation shall be established around the perimeter of the permanent pool to promote plant growth along the shoreline and deter individuals from wading (WEF and ASCE, 2012). The perimeter wetland zone with emergent vegetation shall be at least 10 ft. wide with a water depth of 6 to 12 inches.
- A wetland zone with emergent vegetation is not required in forebays.

Internal Deep Zones

Internal deep zones in the open surface water areas of the wetland and around the outlet:

- Shall be oriented perpendicular to the primary flow path and extend the full width of the practice.
- Shall be 3 to 6 ft. deep, based on dry weather conditions.
- Shall have side slopes 3H:1V or flatter.
- The top width depends on the overall wetland size, however a width of at least 15 ft. is recommended.

Miscellaneous

- If clay, synthetic or plastic liners are used to minimize seepage, cover with gravel, road-base or other material to provide footing and/or utilize other measures to enable egress.
- A minimum 15 ft. wide buffer strip shall be provided around the wetland. The slope of the buffer strip should be 6H:1V or flatter.
- The permanent pools shall have a drain pipe equipped with an adjustable valve that can completely drain the permanent pool. Valve controls shall be located at a point where it will not normally be inundated and can be operated in a safe manner.

Design Considerations

- The area required for a permanent pool is generally one to three percent of its drainage area.
- Stormwater wetlands require groundwater or a dry-weather base flow if the permanent pool elevation is to be maintained year-round.
- The designer should consider the overall water budget to ensure that the baseflow will exceed evaporation, evapotranspiration, and seepage losses (unless the permanent pool is lined).

13.3.6 Storage Pool

A storage pool (detention storage) may be included in the stormwater wetland design.

- The maximum rise in water surface elevation over the permanent pool for the water quality treatment volume shall be 2 ft.

13.3.7 Outlet

Outlets for stormwater wetland can be designed in a wide variety of configurations. Most outlets use modified boxes or riser pipes made of concrete or corrugated metal. These structures can be designed to control different storms using several orifices or pipes; for example, a small inlet to control the water quality volume, an orifice to control a 2-yr storm, and a larger orifice to control a 10-yr storm. This larger flow is usually controlled by stormwater flowing in through the top of the structure. If risers are used, an antivortex design may be necessary for flow entering the top of the pipe. Larger flows are usually handled by an emergency spillway. Because of flow restriction requirements, low flow outlets are often very small. The design must guard against clogging small outlets.

General Requirements

- The stormwater wetland shall be designed with an outlet control system sized to meet the hydrologic requirements.
- Outlet shall be designed to retain floatables, such as debris, oil and grease within the wetland. Acceptable floatables control devices include perforated pipes, skimmers, baffles, inverted pipes and other devices approved by the Department.
- Outlets must be placed near or within the side of the wetland to provide ready maintenance access. Outlets must be constructed of materials that minimize future maintenance requirements.

Orifices

- Orifices shall be 1-inch minimum diameter.
- The outlet shall be designed to resist plugging. Orifices smaller than 4 inches in diameter shall be protected from clogging with an open graded aggregate filter or equivalent.
- When the orifices are in a CMP material the holes shall be pre-drilled prior to galvanizing.

Trash Rack Requirements

Trash rack serve two purposes; (1) to prevent conveying trash and debris downstream, and (2) to act as a safety grate for people and large animals.

- The top of risers and overflow structures shall be equipped with a trash rack.

- Openings shall be a maximum of 4 inches. Recommended opening are 1.5 inches.
- Trash racks are recommended to be installed at a slight angle (approximately 15 degrees) to prevent ice formation and to minimize clogging.

Consider maintenance of the structure and potential access by the public when selecting the type of trash rack. For example, a close mesh grate will be more appropriate in high pedestrian traffic but will require more frequent maintenance as it will catch smaller debris. Trash racks of sufficient size should always be provided on an outlet structure so that they do not interfere with the hydraulic capacity of the outlet.

Piping Downstream of the Outlet Structure

- The minimum pipe size downstream of the flow controls in the practice outlet shall be 12-inch.
- An anti-seepage collar shall be provided on each outlet pipe and watertight joints shall be used on the pipe segment near the anti-seepage collar.
- When connecting to a combined sewer system a backflow preventer and odor trap shall be provided downstream of the outlet structure.

Emergency Overflow

- The emergency spillway elevation shall be set at the elevation of the maximum stormwater wetland practice design volume.
- The emergency spillway shall be designed to pass the maximum design flow tributary to the stormwater wetland practice.
- The emergency overflow must be armored to prevent erosion.
- When an embankment is used around the stormwater wetland practice, the embankment shall be designed to prevent water seepage.

Pumped Outlets

Gravity outlets are preferred over pumped outlets. If a stormwater wetland practice is designed to include a pumped outlet the following requirements must be met:

- A minimum of two pumps should be provided in any pumped outlet system.
- The pumps shall be designed such that the maximum pumping capacity does not exceed the allowable release rate.
- A backup pump shall be provided.

13.3.8 Vegetation

- Plant vegetation is required for all types of wet retention wetlands to control erosion and enhance sediment entrapment.
- A landscaping plan is required for stormwater wetlands, due to the importance of the vegetation to the function of the entire system. The landscape plan shall include at a minimum the following:

- o Existing site conditions and vegetation (e.g. trees 6-inch caliper and larger) that may be affected by the project;
- o Plan view of the open stormwater wetland, including one-foot grading contours at a minimum;
- o Elevations in the stormwater wetland, including the bottom elevation and all the maximum water surface elevations based on the hydrologic requirements;
- o Identification of planting zones based on levels of inundation;
- o Vegetation selection, plant spacing and applicable depths;

- Woody vegetation may not be planted on nor allowed to grow within 15 feet of the toe of an embankment.
- Woody vegetation may not be planted on nor allowed to grow within 25 feet of the emergency overflow.

13.4 Calculations and Sizing

Calculation and sizing of constructed stormwater wetlands follows the same process as discussed in Chapter 7, Detention Practices. Total storage is sized like a detention practice, remembering to disregard the volume of the permanent pools. Stormwater wetlands utilize the same outlet configurations as large detention practices as well.

13.5 Construction Considerations

Construction management of a stormwater wetland is critical. Proper grading of the system is the most important aspect. When the constructed stormwater wetland is adjacent to an existing wetland, a temporary berm or other separation device is needed until the construction is complete.

Hydration of the newly construction stormwater wetland should be carefully managed. Maintaining proper moisture conditions will encourage plant growth and establishment.

Figure 13-9 Wetland restoration signage

Figure 13-10 Attracting wildlife

13.6 Wildlife

Improperly maintained stormwater wetlands provide ideal habitat for urban waterfowl and other nuisance wildlife. Some species such as snakes might be perceived as distasteful to nearby citizens but do not negatively affect the function of the wetland itself. Other species such as Canada geese might negatively affect the wetland by grazing wetland plants, disturbing bottom sediments, and contributing pollutants through fecal matter. Burrowing animals (e.g. beaver and muskrat) may also compromise the geotechnical stability of embankments. Various methods can be used to deter or remove nuisance species from the wetland. Each method should be considered in the context of project objectives, local laws, and stakeholder perception of the nuisance. The most effective method for controlling nuisance waterfowl is to maintain tall vegetation around the entire perimeter of the wetland because waterfowl tend to be wary of tall vegetation for fear of hidden predators. Abundant, diverse vegetation can also provide favorable habitat for dragonflies and other mosquito predators, whereas monocultures of invasive vegetation (such as *Typha* spp. or *Phragmites* spp.) can harbor mosquito larvae in dense mats of roots and detritus (Hunt, Apperson, & Lord, 2005). Where advanced vector control is required, Barrett (2005) recommends introducing *Gambusia affinis* (mosquito fish) at a density of 200 fish per acre of permanent pool (Barrett, 2005). Several references listed below, provide additional information regarding methods of controlling nuisance wildlife:

Managing Waterfowl in Stormwater Ponds:
http://www.clemson.edu/extension/natural_resources/water/stormwater_ponds/nuisance_wildlife/waterfowl/

Goose Control Best Management Practices to Prevent Pollution of Ponds, Streams, and Rivers: http://www.pittsfieldtwp.org/NRC_Goose_Control

Nuisance Wildlife Repellent Handbook:
http://files.dnr.state.mn.us/assistance/backyard/livingwith_wildlife/repellent_handbook.pdf

13.7 Operation and Maintenance

Inspection and maintenance area key to ensure the proper function and aesthetics of stormwater wetlands. Table 13-1 lists specific operation and maintenance tasks.

Table 13-1 Recommended Maintenance

Task	Frequency	Indicator maintenance is needed	Maintenance notes
Forebay inspection	2-4 times/year	Internal erosion or excessive sediment, trash, or debris accumulation	Check for sediment accumulation to ensure that forebay capacity is as designed. Remove any accumulated sediment.
Stormwater wetland inspection	1 time/year	Excessive sediment, trash, and/or debris accumulation in the wetland	Remove any accumulated sediment. Adjacent pervious areas might need to be regraded.
Outlet inspection	2-4 times/year	Accumulation of litter and debris in wetland area, large debris around outlet, internal erosion	Remove litter, leaves, and debris to reduce the risk of outlet clogging and to improve facility aesthetics. Erosion should be repaired and stabilized.
Mowing	2-12 times/year	Overgrown vegetation on embankment or adjacent areas	Frequency depends on location and desired aesthetic appeal.
Embankment inspection	1 time/year	Erosion at embankment	Repair eroded areas and revegetate.
Remove and replace dead vegetation	1 time/year	Dead plants or excessive open areas in wetland	Within the first year, 10% of plants can die. Survival rates increase with time.
Temporary watering	1 time/2-3 days for the first 1-2 months	Wilting plants during drought conditions	Until establishment and in severe drought. Watering after the initial year might be required.
Nuisance wildlife management	Biweekly or as needed	Animals, feces, or burrows evident in or around wetland. Excessive mosquitos.	Maintain diverse vegetated shelf around entire wetland. Eliminate monocultures and replace with diverse, flowing vegetation. Employ qualified wildlife management professionals if needed.
Fertilization	1 time initially	Soil test indicates additional fertilizer is needed	One-time spot fertilization for first year vegetation.

13.8 Design Checklist

13.9 References

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14. Manufactured Treatment Systems

This chapter discusses manufactured treatment systems, which are proprietary commercially available stormwater treatment systems intended to remove non-point source pollutants from the stormwater runoff. The systems generally use some form of filtration, settling, or hydrodynamic separation to remove particulate pollutants from the stormwater runoff.

Contents

14.1 Introduction

Manufactured treatment systems are commercial products intended to remove non-point source pollutants from the stormwater runoff. The methods employed by these products to remove pollutants include settling, filtration, absorption/adsorption, vortex principles, vegetation, and other processes. Two of the most common systems include gravity separation and filtering systems.

Gravity separation is a unit process in which gravity removes settleable solids and associated pollutants, floatables, and dispersed petroleum products *(Minton, 2005)*. The size of a settling basin can be reduced by hydrodynamic separation. Hydrodynamic separators typically use either chambered systems or swirl concentrators to trap and retain sediment from a designed stormwater flow, and use different methods to help prevent the resuspension of sediment during high flow storm events. The retained sediment is removed through periodic maintenance.

Filtering systems typically use a settling chamber and an inert media, such as sand or perlite, to remove specific pollutants. Inert media filtration removes particulate pollutants, whereas sorptive media filtration removes dissolved pollutants. The filtration process is defined by the size of the solids in the water, the water chemistry and the type, size and porosity of the media.

14.2 Approval for Use

14.2.1 Approval Categories

There are two categories of manufactured treatment systems or technologies:

> *Independently Verified Technologies* - patented or other proprietary practices for which pollutant removal, volume reduction and/or peak flow management <u>performance has been verified</u> by a state testing protocol or stormwater trade organization.
>
> *Exploratory Technologies* – patented or other proprietary practices for which <u>performance is not well documented</u> in the engineering community.

The use of exploratory technologies must be first approved by the Department. This typically involves providing relevant product information and conducting pilot testing to assess the pollutant removal, volume reduction and/or peak flow reduction of the technology.

Exploratory technologies may wish to review the requirements in the ASCE publication Stormwater Manufactured Treatment Devices Certification Guidelines (ASCE, 2017). These guidelines provide a framework to verify a technology across a broad range of conditions and the Department will refer to these guidelines as it assesses the performance of manufactured treatment systems.

14.2.2 Independent Testing Protocols

Independently verified technologies have passed an established, independent testing protocol (such as the Water Environment Federation's Stormwater Testing and Evaluation for Products and Practices or STEPP program) which includes an assessment of pollutant removal, volume reduction and/or peak flow reduction.

Independent testing protocols include, but are not limited to:

- New Jersey Department of Environmental Protection (NJDEP) certified and New Jersey Corporation for Advanced Technology (NJCAT) verified
- Water Environment Federation's Stormwater Testing and Evaluation for Products and Practice (STEPP)
- Washington State Department of Ecology - Evaluation of Emerging Stormwater Treatment Technologies using the Technology Assessment Protocol – Ecology (TAPE)
- ASCE *Stormwater Manufactured Treatment Devices Certification Guidelines* (2017).

Note that many testing protocols have focused on total suspended solids (TSS) removal or similar pollutant removal performance evaluations. However, if the technology is claiming volume and/or peak flow reductions, then the technology's volume and/or peak flow reduction performance must also be included in the testing protocol results.

The Department will be primarily using the NJDEP certification for manufactured treatment devices. Certification from other independent testing protocols may be considered, however the NJDEP certification is the preferred approach.

14.2.3 Approval Process

For independently verified technologies, manufacturers shall submit to the Department a letter requesting approval which identifies the water quality unit(s), a copy of the independent testing certification letter, and contact information. The Department will review the independent testing protocol results to assess performance for pollutant removal, volume reduction and/or peak flow reduction. If the technology will be used as a pretreatment device, then only pollutant removal information is necessary. If the technology is acceptable, the Department will issue a certification letter for the manufactured treatment system technology.

For exploratory technologies, the vendor must provide relevant product information and a proposed pilot testing procedure to the Department and request a review. Following the review, the Department will prescribe requisite evaluation and pilot testing procedures and discuss logistical arrangements. After pilot testing is completed, the Department will review the results and determine whether a certification letter for the technology can be issued.

The Department will maintain a list of approved devices.

14.3 Design Standards

Manufactured treatment devices provide water quality treatment to stormwater runoff. The devices are useful for areas that have or are expected to have significant amounts of sediment or debris, or in areas that have specialized treatment needs. Manufactured treatment devices are commonly used as a pre-treatment system before other stormwater management practices. (SEMCOG, 2008)

The following general requirements must be met for using manufactured treatment devices.

- Only use devices approved by the Department.
- The site drainage system shall meet all applicable water quality requirements regardless if the hydraulic capacity of the device is exceeded.
- Selected devices shall be designed such that re-suspension of captured sediment is avoided during storm events that exceed the hydraulic capacity of the device.
- The design shall accommodate the head requirements for the device to work properly, especially when determining the total head requirement for the entire system of stormwater management practices.
- Selection and design of a manufactured treatment device within a stormwater drainage system shall comply with manufacturer recommendations.
- Maintenance access shall be provided to each device.

14.4 Maintenance

Follow the manufacturer's guidelines for operation and maintenance. Consider expected sediment and pollutant load and site conditions.

14.5 References

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