# STORM WATER BEST MANAGEMENT PRACTICES MANUAL

February 2012

Sanitation District No. 1 of Northern Kentucky

City of Florence

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# I. FOREWORD

## PREFACE

Storm water management has entered a new phase in the Northern Kentucky Region for both Sanitation District No. 1 (SD1) and the City of Florence. The requirements for National Pollutant Discharge Elimination System municipal and industrial permits, total maximum daily loads (TMDLs), watershed assessments and the desire to protect human life, property, aquatic habitats and the quality of life in our communities has brought home the pressing need to manage both storm water quantity and quality from our developed and developing areas.

This Manual will help Northern Kentucky move forward with a comprehensive approach to storm water management that integrates site design techniques and storm water quantity and quality considerations to meet the requirements of storm water regulations and utilize storm water as an important resource for our communities. The goal of this Manual is to develop and promote a consistent and effective approach for implementation of storm water management.

## ACKNOWLEDGEMENTS

SD1 and the City of Florence would like to thank the Atlanta Regional Commission, the Georgia Department of Natural Resources-Environmental Protection Division, and 35 cities and counties from across Georgia whose collaborate effort produced the *Georgia Stormwater Management Manual*, from which the background, better site design techniques, and storm water site planning chapters are based. For a complete list of the participants in Georgia's Manual or to view their document visit:

http://www.georgiastormwater.com/

In addition, SD1 and the City of Florence would like to thank Strand Associates, Inc.®, Geosyntec Consultants®, and Sustainable Streams, LLC for working with us to develop the technical standards, selection criteria, and BMP Fact Sheets for this Manual.

Finally, thank you to all those who took the time and effort to provide review comments and constructive suggestions on the draft versions of the Manual.

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# II. BMP MANUAL USER GUIDE

## OBJECTIVE OF THE MANUAL

The objective of the Sanitation District No. 1 of Northern Kentucky (SD1) and the City of Florence Storm Water Best Management Practices (BMP) Manual is to provide guidance on addressing post-construction storm water runoff. The goal is to provide an effective tool for local municipalities and the development community to reduce both storm water quality and quantity impacts, and protect downstream areas and receiving waters. This Manual does not cover construction site sediment and erosion control practices.

## ORGANIZATION OF THE MANUAL

The BMP Manual is designed to provide guidance on the basic principles of effective storm water management. This Manual covers the issues of storm water runoff associated with land development and the need to address storm water quantity and quality, storm water management minimum standards, and guidance on local storm water programs.

In addition, this Manual provides guidance on the techniques and measures that can be implemented to meet a set of storm water management minimum standards for new development and redevelopment projects. The BMP Manual is designed to provide the site designer or engineer, as well as the local plan reviewer or inspector, with all of the information required to effectively address and control both water quality and quantity on a development site. This includes guidance on better site design practices, hydrologic techniques, criteria for the selection and design of structural storm water controls, drainage system design, and construction and maintenance information.

## USERS OF THIS MANUAL

The users of this Manual will be site planners, engineers, contractors, plan reviewers, and inspectors from local government and the development community. SD1 and the City of Florence will use this Manual to review storm water site plans and provide technical advice to meet the post-construction storm water requirements.

Parties involved with site development will utilize the BMP Manual for technical guidance and information on the preparation of storm water site plans, the use of better site design techniques, hydrologic techniques, selection and design of appropriate structural storm water controls, and drainage (hydraulic) design.

## HOW TO USE THIS MANUAL

The following provides a guide to the various chapters of the BMP Manual.

- **Chapter 1 - Introduction**. This section provides a background on SD1, the City of Florence, and conditions of the Northern Kentucky Region.

- **Chapter 2 – The Need for Storm Water Management.** This section provides an overview of the impacts of post-construction storm water runoff.
- **Chapter 3 – Storm Water Management Standards.** This chapter contains the storm water management minimum standards for new development and redevelopment sites. In addition, this chapter explains the sizing criteria for water quality and water quantity post-construction storm water controls.
- **Chapter 4 – Storm Water Better Site Design.** This chapter covers the toolkit of better site design practices and techniques that can be used to reduce the amount of storm water runoff and pollutants generated from a site.
- **Chapter 5 – Storm Water Site Planning.** This chapter outlines the typical contents and procedures for preparing a storm water site plan.
- **Chapter 6 – BMP Selection Guidance.** This chapter contains the information and guidance for the selection and design of structural storm water controls for managing storm water quantity and quality.
- **Chapter 7 – BMP Fact Sheets.** This chapter contains detailed information and design criteria for recommended storm water controls to meet storm water management requirements.
- **Chapter 8 – Example Design Calculations.** This chapter contains four examples utilizing the fact sheet equations for hypothetical development scenarios.
- **Appendix A–BMP Selection Guidance.** This appendix provides additional information on BMP performance from the International Stormwater BMP Database.
- **Appendix B–Bioretention Soil Mix.** This appendix provides guidance and specifications for general bioretention soil mixes.
- **Appendix C–Plant Selections.** This appendix provides information and specifications for native plant species for bioretention practices.
- **Appendix D–Soil Testing Procedures.** This appendix provides information and recommendations for different soil testing procedures.
- **Appendix E–Outlet Design Guidance.** This appendix provides outlet design guidance for sizing outlet structures for meeting water quality drain time requirements.
- **Appendix F–Example Hydraulic Control Structure Schematics.** This appendix provides example hydraulic control schematics.
- **Appendix G–BMP Inspection and Maintenance Checklists.** This appendix provides BMP inspection and maintenance checklists for the BMP types included in this manual.
- **Appendix H–Post-Construction Storm Water Controls - Maintenance Agreement.** This appendix contains SD1's Operation and Maintenance Agreement.

## REGULATORY STATUS OF THE MANUAL

The methods and techniques outlined in this BMP Manual are intended as a tool to assist developers and engineers to meet the requirements of SD1's Storm Water Rules & Regulations and Boone County Subdivision Regulations. The BMP Manual may be amended from time to time as required.

SD1: The BMP Manual does not include regulations within the meaning of KRS 220.320. Regulations are set out in SD1's Storm Water Rules & Regulations. Where there exists any conflict between the Storm Water Rule & Regulations and the BMP Manual, the Storm Water Rules & Regulations shall prevail.

City of Florence: Regulations are set forth in the Boone County Subdivision Regulations. Where there exists any conflict between the Boone County Subdivision Regulations and the BMP Manual, the Boone County Subdivision Regulations shall prevail.

## HOW TO FIND THE MANUAL ON THE INTERNET

The Storm Water BMP Manual is available in Adobe Acrobat PDF document format for download at the following Internet addresses:

http://www.sd1.org

http://www.florence-ky.gov/storm_water.asp

## CONTACT INFORMATION

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# CHAPTER 1
# INTRODUCTION

## 1.1 LOCAL PARTICIPATING AGENCIES

Northern Kentucky, which is located just south of Cincinnati, Ohio, consists of 37 cities and three counties (Boone, Campbell and Kenton). The population of the three county region is approximately 365,000 (U.S. Census Bureau, 2009 Population Estimates). Northern Kentucky proudly serves as the home for prospering businesses such as Toyota, Fidelity Investments and the Northern Kentucky Greater Cincinnati Airport.

### *1.1.1 Sanitation District No. 1*

Sanitation District No. 1 (SD1) is the second largest public sewer utility in Kentucky serving 33 communities, including unincorporated portions of Boone, Campbell and Kenton Counties, with an overall service area of approximately 223 sq miles.

SD1 was established in 1946 by the Division of Sanitary Engineering of the Kentucky Department of Health pursuant to an amendment of Chapter 220 of the Kentucky Revised Statutes (KRS 220). Prior to 1946, a small system of sewer lines already existed in Northern Kentucky; however, the region was still in need of proper wastewater treatment. The amendment to KRS 220 gave SD1 authority to prevent and correct the pollution of streams, regulate the flow of streams for sanitary purposes, clean and improve stream channels for sanitary purposes, and collect and dispose of sewage and other liquid wastes produced throughout the established service area. It also granted SD1 authority to construct sewers, trunk sewers, laterals, intercepting sewers, siphons, pump stations, treatment and disposal works and other appropriate facilities.

It was not until legislation was adopted in 1998 by the Kentucky General Assembly that SD1 was granted authority to regulate and finance storm water facilities within the designated service area. In response to requests from Northern Kentucky communities, SD1 accepted the responsibility to develop and implement a regional storm water management program to comply with U.S. EPA's 1999 National Pollutant Discharge Elimination System (NPDES) Phase II Rule. The Phase II Rule is briefly discussed in Section 1.2.

SD1 is responsible for all of the sanitary sewer systems in Northern Kentucky, with the exception of the City of Florence and the City of Walton; SD1 also manages the Phase II Rule for all Northern Kentucky designated municipalities, with the exception of the City of Florence. SD1 maintains more than 1,600 miles of sanitary sewers, 145 wastewater pumping stations, 15 flood pump stations, 8 package treatment plants, two major wastewater treatment plants, 400 miles of storm sewers and over 28,800 storm sewer structures.

### *1.1.2 City of Florence*

The City of Florence is the eighth largest city in Kentucky with a population of over 28,000 and is located in the 71st most rapidly growing county (Boone County) in the country. Florence is home to a large number of employers in the industrial fields and many hotels, restaurants and retail shopping areas. Florence is a full service community offering police, fire, EMS/ALS, water, sewer, public services, recreation, and administrative support services. Florence is 10.31 sq miles in area.

The Public Service Department is responsible for all maintenance, cleaning, rehabilitation and inspection for the sanitary sewer lines within the City of Florence sservice area. The sanitary sewer system includes approximately 122 miles of sanitary sewers and 13 pump stations. The installation of the sanitary sewer system in the City of Florence started in the early 1950’s and is in constant need of maintenance and repair. The installation of the storm sewer system in the City of Florence started in the early 1950’s. The Public Service Department is responsible for the maintenance and management of the storm system including 142 miles of storm sewers, over 3,300 storm sewer structures, 12 City maintained detention/retention basins and 196 privately maintained detention basins. The City’s first storm water master plan was completed in 1990 and subsequent updates were completed in 2005. As part of the 2005 storm water master plan update, the City clearly defined “Waters of Florence “ in order to delineate private and public storm water issues. The City of Florence has been designated by KDOW as a storm water Phase II community. The Phase II Rule is briefly discussed below.

## 1.2 PHASE II REGULATIONS

U.S. EPA’s National Pollutant Discharge Elimination System (NPDES) Phase II Rule applies to operators of regulated small municipal separate storm sewer systems (MS4s) serving a population of less than 100,000 people in urbanized areas. The final rule required all MS4s located within urbanized areas, as defined by the Bureau of the Census, to comply with the Phase II Storm Water Regulations. The Final Rule requires that the NPDES permitting authority, Kentucky Division of Water (KDOW), develop and apply designation criteria to make a final determination of which communities are required to comply with this regulation.

KDOW has designated over 30 communities in Boone, Campbell and Kenton Counties (including the counties themselves) as Phase II communities that must comply with the NPDES regulations, which have been adopted by KDOW at 401 KAR 5:060 Section 12. The US Environmental Protection Agency (EPA) has delegated responsibility for the MS4 program to the Kentucky Energy and Environment Cabinet, of which KDOW is a part.

The Phase II communities in Northern Kentucky, as owners/operators of small MS4s, are required to reduce the discharge of pollutants from the MS4 to waters of the Commonwealth and the United States to the “maximum extent practicable” to protect water quality. The Phase II Rule outlines six minimum control measures to help MS4s address this goal. The six minimum controls as defined by EPA are as follows:

- Public Education and Outreach;
- Public Involvement and Participation;
- Illicit Discharge Detection and Elimination;
- Control of Construction Site Runoff;
- Post-Construction Storm Water Management; and,
- Pollution Prevention and Good Housekeeping.

On behalf of 33 Phase II communities, in Northern Kentucky, SD1 manages the Phase II regulations. The City of Florence conducts their own Phase II compliance efforts. SD1 and the City of Florence continue to partner on various storm water initiatives to help improve and protect water quality in Northern Kentucky. SD1 and the City of Florence developed this manual with the goal of providing an effective tool for local municipalities and the development community to reduce both storm water quality and quantity impacts, protect downstream areas and receiving waters and comply with increasingly more stringent post-construction requirements.

## 1.3 LOCAL CONDITIONS

### *1.3.1 Geography*

Boone, Campbell and Kenton counties are the three most northern counties in Kentucky, see Figure 1.3-1. The three-county area, including more than 30 cities, is 25 to 39 miles wide from east to west and 20 to 22 miles long from north to south. The total land area is 360,320 acres or about 563 square miles. The area is a part of the Greater Cincinnati Metropolitan Area.

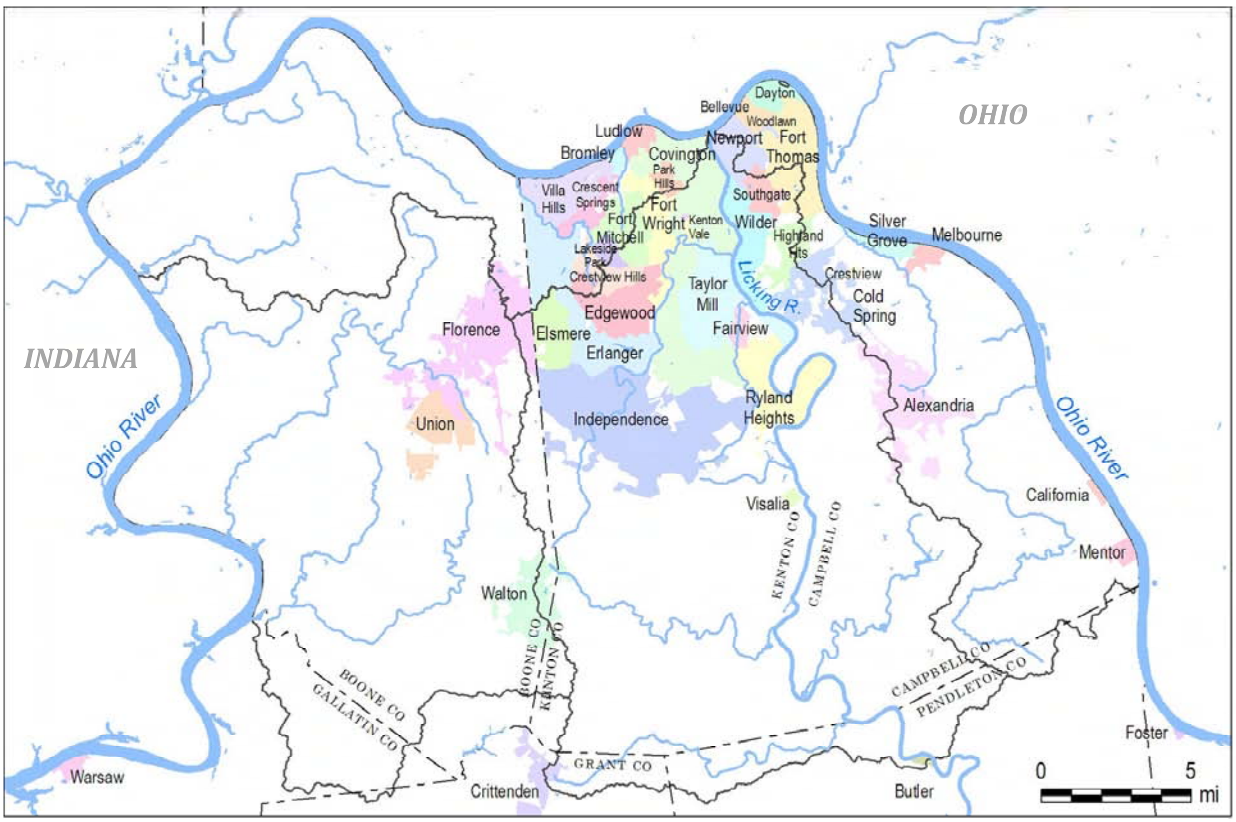


Figure 1.3-1 Northern Kentucky Three County Region, Including Thirty-Seven Cities

The counties, part of the Kentucky Bluegrass Region, are bounded by the Ohio River to the north, east and west. The Licking River separates Campbell and Kenton Counties.

Boone County, with an approximate population of 118,576 is the western-most county of the three Northern Kentucky counties. It is generally bounded by the Ohio River on its north and west side and by the Interstate 75/71 on most of its east side. Its topography is the most level of the region’s varying topography. Boone County is the location of the Cincinnati/Northern Kentucky International Airport. Topography, airport location and interstate access are factors that have made Boone County the fastest growing county in the region and one of the fastest growing counties in the state. New businesses and new residents have changed the socioeconomic structure of this county that, until only recently, was a rural, agriculturally-based community. Recent developments in Boone County have been located around the airport or adjacent to the three cities of Florence (28,381), Union (3,710) and Walton (3,119). Future growth will be in the central and southern parts of the county.

Campbell County, with an approximate population of 88,423 is the eastern-most county of the three counties. The county is bounded by water on three sides--the Ohio River on the north and east and the Licking River on the west. The historic riverfront communities of Newport, Bellevue and Dayton enjoy views of the Cincinnati skyline and activities on the Ohio River. Marinas, restaurants and other commercial projects have located on the south shore of the Ohio River in Campbell County to take advantage of the scenery. The topography of Campbell County that creates the vistas for residential use limits other commercial development to valley floors or upland areas in the southern part of the county. A major institution in Campbell County is Northern Kentucky University, the fastest growing senior college in the state system. Several of the more populous of Campbell County's 15 cities and towns are Newport (15,863), Fort Thomas (15,271) and Alexandria (8,519).

Kenton County, with an approximate population of 158,729 is the central of the three counties. It is generally bounded by the Ohio River on the north side, by the Licking River on the east side and by Interstate 75/71 on most of its west side. Kenton County is the more densely populated of the three counties, beginning along the historic waterfront area of Covington and moving south toward the circumferential Interstate 275. The topography of Kenton County is fairly steep just south of the Ohio River basin, but moderately to gently rolling terrain dominates its center and southern sections. The economy and residential sectors of Kenton County are more mature in comparison to the recent vintages of Boone County. Future growth in Kenton County will be predominantly in the southwest part of the county. Several of the more populous of Kenton's 19 cities and towns are Covington (43,082), Independence (22,105) and Erlanger (17,259).

### *1.3.2 Climate and Hydrology*

The climate in the Northern Kentucky Area is continental with a wide range of temperatures from winter to summer. Weather movement and wind direction is generally from southwest to northeast. Summers are warm and humid with 90-degree temperatures or higher occurring about 20 days each year. Winters are moderately cold with frequent periods of cloudiness; maximum snowfall occurs during January. The freeze-free period lasts, on the average, 187 days from mid-April to the latter part of October. Temperature and precipitation patterns may be summarized as follows, in Table 1.3-1:

Table 1.3-1 Northern Kentucky Temperature and Precipitation Patterns

| TEMPERATURE | |
|---|---|
| Normal (30-year record) | 54.2 degrees F |
| Average annual (2007) | 56.1 degrees F |
| Record highest, July 1988 (46-year record) | 103.0 degrees F |
| Record lowest, January 1977 (46-year record) | -25.0 degrees F |
| PRECIPITATION | |
| Normal (Year 1949 - 2011) | 40.1 inches |
| Record total precipitation (2011) | 73.26 inches |
| Sources: Weather Underground; ThinkKentucky.com | |

### *1.3.3 Water Resources*

The green rolling hills of Kentucky rise from the banks of the Ohio River, the region's most prominent and historical waterway. The Ohio River is 981 miles long, stretching from the Allegheny and Monongahela rivers in Pittsburgh to the Mississippi River in Cairo, Illinois. The river borders six states: Pennsylvania, Ohio, West Virginia, Kentucky, Indiana and Illinois. This river is what separates Northern Kentucky from the region's largest city - Cincinnati, Ohio.

The Ohio River Basin is one of the most populated and industrialized regions in the United States, with more than 25 million people living within its watershed. The Ohio River is also a source of drinking water for thousands of communities. More than three million people in the Greater Cincinnati area alone depend on the Ohio River for drinking water. Urban and agricultural runoff, abandoned mines, industrial waste and sewage are threats to water quality in the Ohio River.

Northern Kentucky has 13 primary watersheds (see Figure 1.3-2) that eventually feed into the Ohio River (from east to west): Twelvemile Creek, Fourmile Creek, Taylor Creek, Licking River (Banklick Creek and Threemile Creek - Licking River tributaries), Pleasant Run Creek, Dry Creek, Elijahs Creek, Sand Run, Woolper Creek, Gunpowder Creek, and Big Bone Creek.

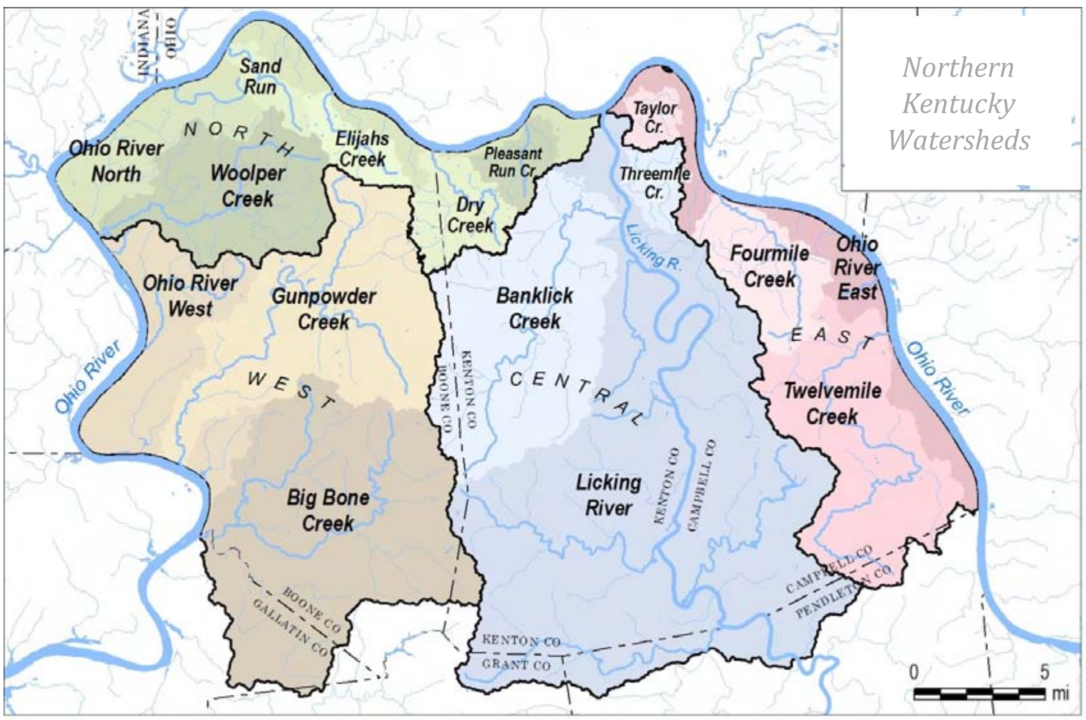



Figure 1.3-2 Northern Kentucky's Thirteen Primary Watersheds

The Licking River begins in the mountains of Eastern Kentucky and meanders northwest until it meets the Ohio River. The Licking River Watershed drains roughly 3,600 square miles (about 10%) of Kentucky. More than 250,000 Kentuckians rely on the Licking River for drinking water. Protection of these resources is vital to the environment, public health and the economy.

The Licking River watershed, as well as the other watersheds in Northern Kentucky, is included in

KDOW's Licking River Basin management unit. Most of these local watersheds are influenced by the impacts of civilization ranging from rural agricultural activities to suburban and commercial/industrial pressures to highly impervious urbanized areas. In order to keep track of the quality of Kentucky's streams and lakes, KDOW updates the Kentucky 303(d) List, which identifies streams and lakes as impaired for identified pollutants or as not meeting one or more of the water quality standards, every two years. Refer to the Energy and Environment Cabinet, Division of Water website for the latest 303(d) to identify impaired waterbodies in Northern Kentucky (http://water.ky.gov/ ).

In general, impaired uses for these waterbodies include primary contact recreation and warm water aquatic habitat as either partial supporting or nonsupporting. Typical pollutants of concern include bacteria, nutrients and sediments.

Also, there are many stream segments in Northern Kentucky that KDOW has classified as special use waters which include exceptional waters, reference reach waters and outstanding state resource waters. Refer to the Energy and Environment Cabinet, Division of Water website for the latest special use waters in Northern Kentucky. (http://water.ky.gov/)

### *1.3.4 Geology*

Northern Kentucky lies within the Till Plains section of the Central Lowland physiographic province. This province is characterized by structural and sedimentary basins, domes and arches which came into existence throughout Paleozoic time. Among these features, the Cincinnati geoanticline, or "Cincinnati Arch," is structurally significant. The Northern Kentucky Area is at the crest of this arch. This axis passes through Kenton County. The bedrock underlying the region is composed of shale and fossiliferous limestone of Middle and Late Ordovician age. It outcrops on steep valley walls and at numerous waterfalls.

The area is part of an upland plain rising some 960 feet above sea level. All of the area drains into the Ohio River and its tributaries. The Ohio River crosses the area in a valley some 500 feet below the general level of the plain.

The main local physiographic features are gently rolling glacial uplands, steep hillsides along the major streams and flood plains. See Figure 1.3-3 for an overview of the geology in the Northern Kentucky Region.

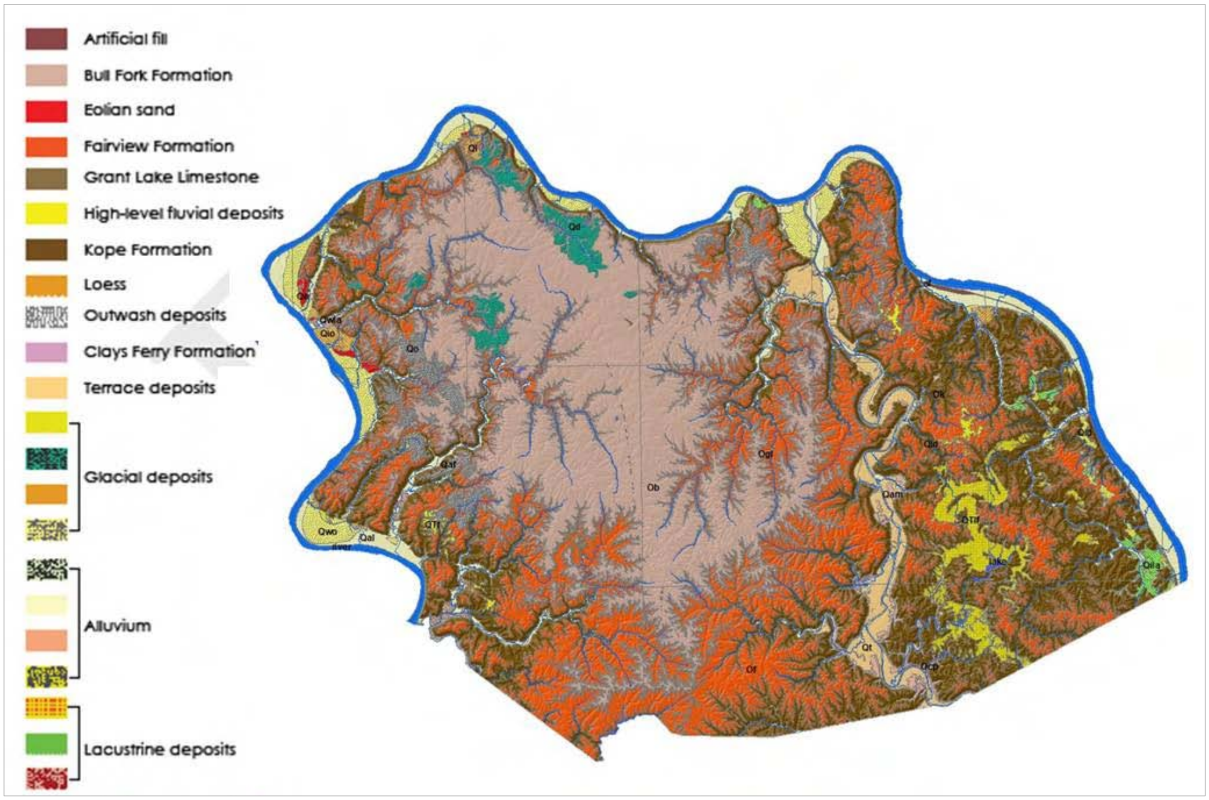



Figure 1.3-3 Overview of Northern Kentucky's Geology

### *1.3.5 Soils*

Roughly half of the Northern Kentucky Area is in the Hills of the Bluegrass, heavy clay based soil, and the other half, the more nearly level part, is in the Outer Bluegrass. The Hills of the Bluegrass are rather steep areas. The soils are underlain by thin beds of limestone and calcareous shale. The Outer Bluegrass area is mostly rolling to undulating.

The soils along the Ohio River are generally loamy or sandy and better drained than the soils along the Licking River and the small creeks. Many soils along the Licking River, along small creeks and in central Campbell County were formed in slack water and are fine-textured sediments. In the upland areas between the Ohio and Licking River valleys, the soils are of glacial, colluvial, alluvial, and residual origin. Glacial soils include those deposited by melting ice, temporary glacial lakes, and wind following glacial retreat. They also include soils that have been reworked in place by the advancement of over-riding glaciers. Colluvial soils are formed by downslope transport of soil and bedrock materials under the influence of gravity. Alluvian soils are deposited in stream valleys by moving water. Residual soils are formed by in situ weathering of the underlying shale and limestone bedrock.

The National Resource Conservation Service (NRCS) has a classification system for soils which denotes the storm water runoff potential of the soil; these are called the hydrologic soil groups (HSG). Group A has the lowest runoff potential as these soils consist of sandy soils and high infiltration rates. Group B has a moderate runoff potential with soils that have moderately fine to moderately coarse textures. Group C soils are typically sandy clay loam soils having moderately fine to fine textures and a low infiltration capacity. Group D soils, having a very high runoff potential, are comprised mostly of clay. Figure 1.3-4 depicts the HSG for the soils in Northern Kentucky. Those areas that are unclassified are the urban land and quarries within the area. Nearly half of all Northern Kentucky soil is classified as Group C due to the amount of clay that is found in the soils. In addition, the urban core results in a soil type of unclassified due to the level of development and infill in the area.

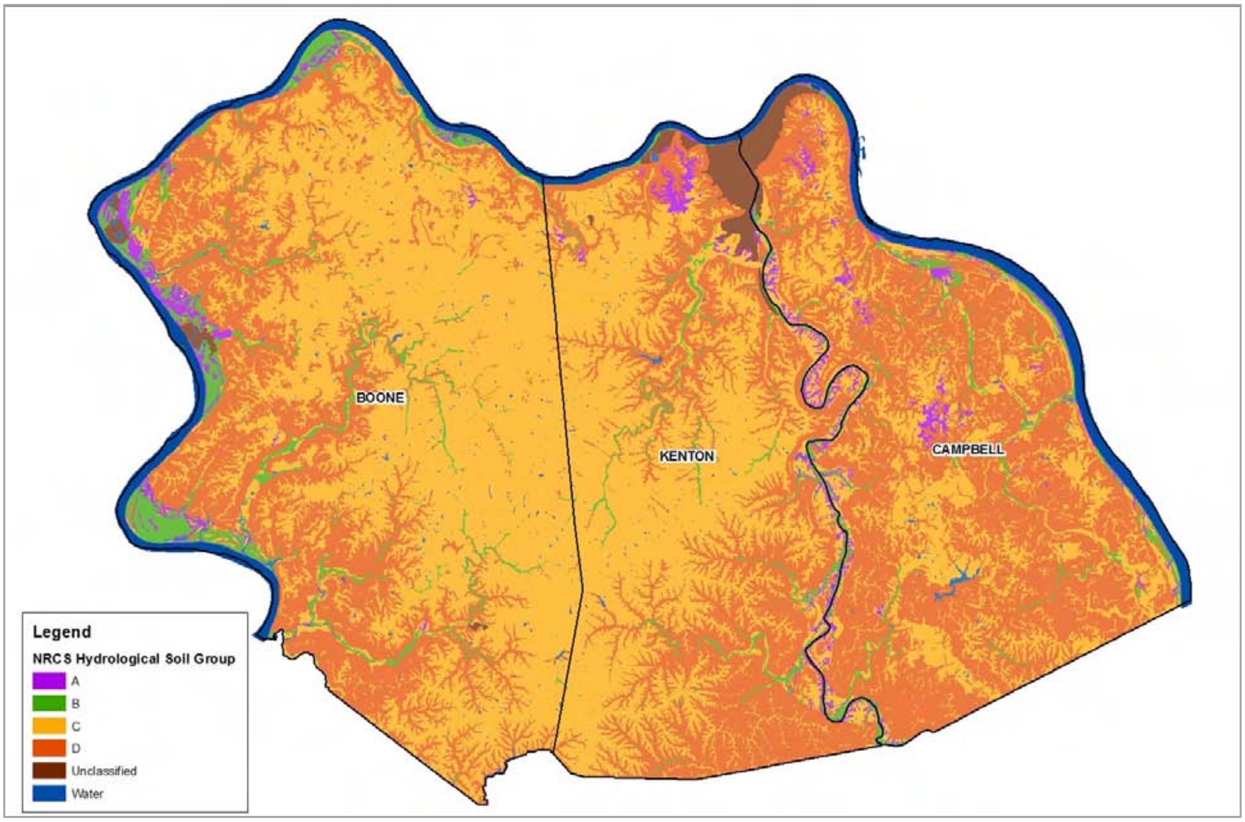



Figure 1.3-4 Overview of Northern Kentucky's Hydrologic Soil Groups

# CHAPTER 2
# THE NEED FOR STORM WATER MANAGEMENT

## 2.1 IMPACTS OF DEVELOPMENT AND STORM WATER RUNOFF

Disturbance to natural landscapes as a result of human activity is among the most important issues facing storm water management. Across the nation, the effects of these changes have been well documented, and recently these changes have been assessed for Northern Kentucky. Land development changes not only the physical, but also the chemical and biological conditions of Northern Kentucky's waterways and water resources. This chapter describes the changes that occur due to development and the resulting storm water runoff impacts.

### *2.1.1 Development Changes Land and Runoff*

When land is developed, the hydrology, or the natural cycle of water is disrupted, altering the delivery of water and sediment to local waterways. Clearing removes the vegetation that intercepts, slows and returns rainfall to the air through evaporation and transpiration. Grading flattens hilly terrain and fills in natural depressions that slow and provide temporary storage for rainfall. Rainfall that once seeped into the ground now runs off the surface. The addition of buildings, roadways, parking lots and other surfaces that are impervious to rainfall further reduces infiltration and increases runoff.

Figure 2.1-1 Clearing and grading alter the hydrology of the land

Depending on the magnitude of changes to the land surface, both the total runoff volume and magnitude can increase dramatically. These changes not only increase the total volume of runoff, but also accelerate the rate at which runoff flows across the land. This effect is further exacerbated by drainage systems such as gutters, storm sewers and lined channels that are designed to quickly carry runoff to rivers and streams.

Figure 2.1-2 Impervious Cover Increases Runoff Peak Flows and Volumes While Reducing Recharge

Development and impervious surfaces also reduce the amount of water that infiltrates into the soil and groundwater, thus reducing the amount of water that can recharge aquifers and feed streamflow during periods of dry weather.

Finally, development and urbanization affect not only the quantity of storm water runoff, but also its quality. Development increases both the concentration and types of pollutants carried by runoff. As it runs over rooftops and lawns, parking lots and industrial sites, storm water picks up and transports a variety of contaminants and pollutants to downstream waterbodies. The loss of the original topsoil and vegetation removes a valuable filtering mechanism for storm water runoff.

The cumulative impact of development and urban activities, and the resultant changes to both storm water quantity and quality over the entire land area that drains to a stream, river, lake or reservoir (known as its watershed) directly impacts the conditions of the waterbody. Land development within a watershed has a number of direct impacts on downstream waters and waterways. These impacts include:

- Changes to stream flow;
- Changes to stream geometry;
- Degradation of aquatic habitat; and,
- Water quality impacts.

The remainder of this section discusses these impacts and why effective storm water management is needed to address and mitigate them.

### *2.1.2 Changes to Stream Flow*

Land development alters the hydrology of watersheds and streams by disrupting the natural water cycle. This results in:

- Increased Runoff Volumes – Land surface changes can dramatically increase the total volume of runoff generated in a developed watershed.
- Increased Peak Runoff Discharges – Increased peak discharges for a developed watershed can be two to five times higher than those for an undisturbed watershed.
- Greater Runoff Velocities – Impervious surfaces and compacted soils, as well as improvements to the drainage system such as storm drains, pipes and ditches, increase the speed of storm water runoff.
- Timing – As runoff velocities increase, it takes less time for water to reach a stream or other waterbody.
- Increased Frequency of Bankfull and Near Bankfull Events – Increased runoff volumes and peak flows increase the frequency and duration of smaller bankfull and near bankfull events which are the primary channel forming events.
- Increased Flooding – Increased runoff volumes and peaks also increase the frequency, duration and severity of out-of-bank flooding.
- Lower Dry Weather Flows (Baseflow) – Reduced infiltration of storm water runoff causes streams to have less baseflow during dry weather periods and reduces the amount of rainfall recharging groundwater aquifers.

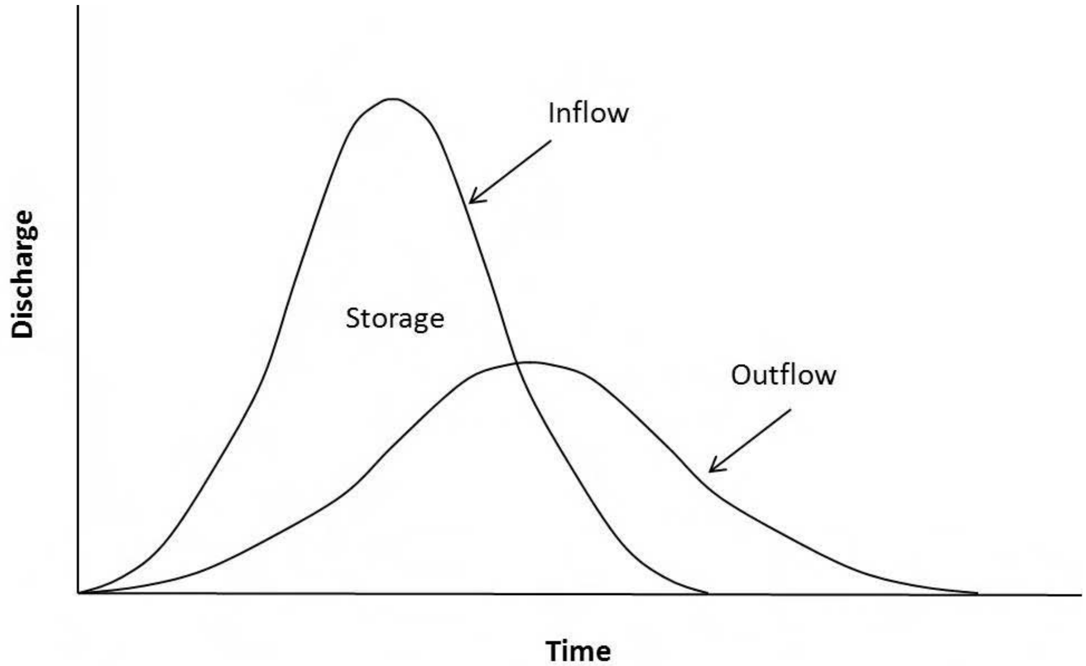



Figure 2.1-3 Hydrograph under Pre- and Post-Development Conditions

### *2.1.3 Changes to Stream Geometry*

The changes in the rates and amounts of runoff from developed watersheds directly affect the morphology, or physical shape and character, of Northern Kentucky's streams and rivers. Some of the impacts due to development include:

- Stream Widening and Bank Erosion – Stream channels widen to accommodate and convey the increased runoff and higher stream flows from developed areas. More frequent small and moderate runoff events undercut and scour the lower parts of the streambank, causing the steeper banks to slump and collapse during larger storms. Higher flow velocities also increase streambank erosion rates causing a stream to widen many times its original size due to post-development runoff.
- Stream Downcutting – Another way that streams accommodate higher flows is by downcutting their streambed. This causes instability in the stream profile, or elevation along a stream's flow path, which increases velocity and triggers further channel erosion both upstream and downstream.
- Loss of Riparian Tree Canopy – As streambanks are gradually undercut and slump into the channel, the trees that had protected the banks are exposed at the roots. This leaves them more likely to be uprooted during major storms, further weakening bank structure. These uprooted trees can also have damming effect on the stream, creating additional flooding issues where none existed before.
- Changes in the Channel Bed Due to Sedimentation – Due to channel erosion and other sources upstream, sediments are deposited in the stream as sandbars and other features, covering the channel bed, or substrate, with shifting deposits of mud, silt and sand.
- Increase in the Floodplain Elevation – To accommodate the higher peak flow rate, a stream's floodplain elevation typically increases following development in a watershed. This problem is compounded by building and filling in floodplain areas, which cause flood heights to rise even further. Property and structures that had not previously been subject to flooding may now be at risk.

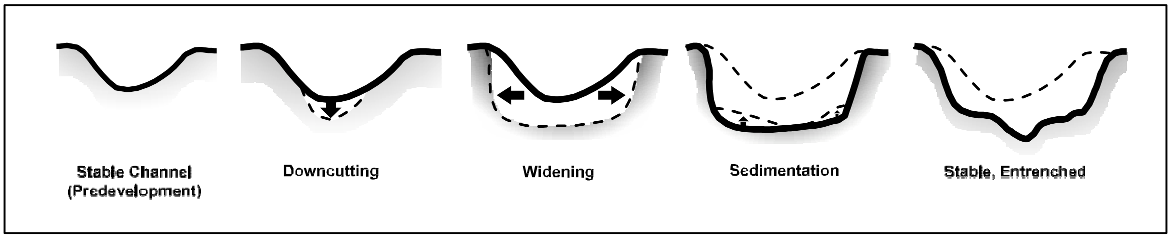



Figure 2.1-4 Changes to a Stream's Physical Character Due to Watershed Development

### *2.1.4 Impacts to Aquatic Habitat*

Along with changes in stream hydrology and morphology, the habitat value of streams diminishes due to development in a watershed. Impacts on habitat include:

- Degradation of Habitat Structure – Higher and faster flows due to development can scour channels and wash away entire biological communities. Streambank erosion and the loss of riparian vegetation reduce habitat for many fish species and other aquatic life, while sediment deposits can smother bottom-dwelling organisms and aquatic habitat.
- Loss of Pool-Riffle Structure – Streams draining undeveloped watersheds often contain pools of deeper, more slowly flowing water that alternate with "riffles" or shoals of shallower, faster flowing water, both of which provide valuable habitat for fish and aquatic insects. As a result of the increased flows and sediment loads from watersheds, pools and riffles begin to disappear and are often replaced with more uniform, and often shallower, streambeds that provide less varied aquatic habitat.
- Reduce Baseflows – Reduced baseflows due to increased impervious cover in a watershed and the loss of rainfall infiltration into the soil and water table adversely affect in-stream habitats, especially during periods of drought.
- Increased Stream Temperature – Runoff from warm impervious areas, storage in impoundments, loss of riparian vegetation and shallow channels can all cause an increase in temperature in urban streams. Increased temperatures can, among other things, reduce dissolved oxygen levels, or disrupt the food chain. Many aquatic species are very sensitive to both in stream temperature and dissolved oxygen levels.
- Decline in Abundance and Biodiversity – When there is a reduction in various habitats and habitat quality, both the number (abundance) and the variety (diversity) of organisms (wetland plants, fish, macroinvertebrates, etc.) are also reduced. Sensitive species disappear and are replaced by those organisms that are more tolerant to the poorer conditions. The diversity and composition of the benthic (streambed) insect community have been used for decades to evaluate the quality of streams because of their sensitivity to change, as well as forming the base of the stream food chain. Altering this foundation of the stream community will ultimately alter the entire ecological integrity.

Fish and other aquatic organisms are impacted not only by the habitat changes brought on by increased storm water runoff quantity, but are often also adversely affected by water quality changes due to development and resultant land use activities in a watershed.

### *2.1.5 Water Quality Impacts*

Nonpoint source (NPS) pollution, a primary cause of water quality impairment, is largely caused by polluted or poorly managed storm water runoff.  NPS pollution comes from many diffuse or scattered sources—most of which are the result of human activities within a watershed. Development concentrates and increases the amount of these nonpoint source pollutants. As storm water runoff moves across the land surface, it picks up and carries away both natural and human-made pollutants, depositing them into Northern Kentucky's streams, rivers, lakes, wetlands, marshes, and underground aquifers. Nonpoint source pollution is a leading source of water quality degradation in Northern Kentucky.

The water quality of a watershed may be impacted when new and redevelopment occurs. Erosion from poorly maintained construction sites and other disturbed areas contribute large amounts of sediment to streams. As construction and development proceed, impervious surfaces replace the natural land cover and pollutants from human activities begin to accumulate on these surfaces. During storm events, these pollutants are then washed off into the streams.

Due to the magnitude of the problem, it is important to understand the nature and sources of storm water pollution. Table 2.1-1 summarizes the major storm water pollutants and their effects. Some of the most frequently occurring pollution impacts and their sources for urban streams are:

- Reduced Oxygen in Streams – The decomposition process of organic matter uses up dissolved oxygen (DO) in the water, which is essential to aquatic life. As organic matter is washed off by storm water, dissolved oxygen levels in receiving waters can be rapidly depleted. If the DO deficit is severe enough, fish kills may occur and stream life can weaken and die. In addition, oxygen depletion can affect the release of toxic chemicals and nutrients from sediments deposited in a waterway.

All forms of organic matter in storm water runoff, such as leaves, grass clippings and pet waste contribute to the problem.

ABLE 2.1-1 SUMMARY OF URBAN STORM WATER POLLUTANTS

| CONSTITUENTS | EFFECTS |
|---|---|
| Sediments—Suspended Solids, Dissolved Solids | Stream turbidity<br>Habitat changes<br>Recreation/aesthetic loss<br>Contaminant transport<br>Filling of lakes and reservoirs |
| Nutrients—Nitrate, Nitrite, Ammonia, Organic Nitrogen, Phosphate, Total Phosphorus | Algae blooms<br>Eutrophication<br>Ammonia and nitrate toxicity<br>Recreation/aesthetic loss |
| Bacteria—Total and Fecal Coliforms, Fecal Streptococci Viruses, E. coli, Enterococci | Ear/Intestinal infections<br>Shellfish bed closure<br>Recreation/aesthetic loss |
| Organic Matter—Vegetation, Sewage, Other Oxygen Demanding Materials | Dissolved oxygen depletion<br>Odors<br>Fish kills |

| | |
|---|---|
| Toxic Pollutants—Heavy Metals (cadmium, copper, lead, zinc), Organics, Hydrocarbons, Pesticides/Herbicides | Human & aquatic toxicity<br>Bioaccumulation in the food chain |
| Thermal Pollution | Dissolved oxygen depletion<br>Habitat changes |
| Trash and debris | Recreation/aesthetic loss |

- Nutrient Enrichment – Runoff from developed watersheds contains increased nutrients, such as nitrogen or phosphorus compounds. Increased nutrient levels are a problem as they promote weed and algae growth in lakes and streams. Algae blooms block sunlight from reaching underwater grasses and deplete oxygen in bottom waters. In addition, nitrification of ammonia by microorganisms can consume dissolved oxygen, while nitrates can contaminate groundwater supplies. Sources of nutrients in the developed environment include washoff of fertilizers and vegetative litter, animal wastes, sewer overflows and leaks, septic tank seepage, detergents, and the dry and wet fallout of materials in the atmosphere.
- Microbial Contamination – The level of bacteria, viruses and other microbes found in urban storm water runoff often exceeds public health standards for water contact recreation such as swimming and wading. Microbes can also contaminate shellfish beds, preventing their harvesting and consumption, as well as increasing the cost of treating drinking water. The main sources of these contaminants are sewer overflows, septic tanks, pet waste, and wildlife.
- Hydrocarbons – Oils, greases and gasoline contain a wide array of hydrocarbon compounds, some of which have shown to be carcinogenic, tumorigenic and mutagenic in certain species of fish. In addition, in large quantities, oil can impact drinking water supplies and affect recreational use of waters. Oils and other hydrocarbons are washed off roads and parking lots, primarily due to engine leakage from vehicles. Other sources include the improper disposal of motor oil in storm drains and streams, spills at fueling stations and restaurant grease traps.
- Toxic Materials – Besides oils and greases, storm water runoff can contain a wide variety of other toxicants and compounds, including heavy metals such as lead, zinc, copper, and cadmium, as well as organic pollutants such as pesticides, polychlorinated biphenyl (PCBs), and phenols. These contaminants are of concern because they are toxic to aquatic organisms and can bioaccumulate in the food chain. In addition, they also impair drinking water sources and human health. Many of these toxicants accumulate in the sediments of streams and lakes.

Sources of these contaminants include industrial and commercial sites, urban/suburban surfaces such as rooftops and painted areas, vehicles and other machinery, improperly disposed household chemicals, landfills, hazardous waste sites and atmospheric deposition.

- Sedimentation – Eroded soils are a common component of storm water runoff and are a pollutant in their own right. Excessive sediment can be detrimental to aquatic life by interfering with photosynthesis, respiration, growth and reproduction. Sediment particles transport other pollutants that are attached to their surfaces, including nutrients, trace metals and hydrocarbons. High turbidity due to sediment increases the cost of treating drinking water and reduces the value of surface waters for industrial and recreational use. Sediment also fills ditches and small streams and clogs storm sewers and pipes, causing flooding and property damage. Sedimentation can reduce the capacity of reservoirs and lakes, block navigation channels, and fill harbors. Erosion from construction sites, exposed soils, street runoff, and streambank erosion are the primary sources of sediment in runoff.

- Higher Water Temperatures – As runoff flows over impervious surfaces such as asphalt and concrete, it increases in temperature before reaching a stream or pond. Water temperatures are also increased due to shallow ponds and impoundments along a watercourse as well as fewer trees along streams to shade the water. Since warm water can hold less dissolved oxygen than cold water, this "thermal pollution" further reduces oxygen levels in depleted urban streams. Temperature changes can severely disrupt certain aquatic species, such as stoneflies, which can survive only within a narrow temperature range.
- Trash and Debris – Considerable quantities of trash and other debris are washed through storm drain systems and into streams and lakes. The primary impact is the creation of an aesthetic "eyesore" in waterways and a reduction in recreational value. In smaller streams, debris can cause blockage of the channel, which can result in localized flooding and erosion.

### *2.1.6 Storm Water Hotspots*

Storm water hotspots are areas of the urban landscape that often produce higher concentrations of certain pollutants, such as hydrocarbons or heavy metals, than are normally found in urban runoff. These areas merit special management and the use of specific pollution prevention activities and/or structural storm water controls. Examples of storm water hotspots include:

- Gas / fueling stations;
- Vehicle maintenance areas;
- Vehicle washing / steam cleaning;
- Auto recycling facilities;
- Outdoor material storage areas;
- Loading and transfer areas;
- Landfills;
- Construction sites;
- Industrial sites; and,
- Industrial rooftops.

### *2.1.7 Effects on Lakes and Reservoirs*

Storm water runoff into lakes and reservoirs can have some unique and negative effects. A notable impact of urban runoff is the filling in of lakes and reservoirs with sediment. Another significant water quality impact on lakes related to storm water runoff is nutrient enrichment. This can result in the undesirable growth of algae and aquatic plants. Lakes and reservoirs do not flush contaminants as quickly as streams and act as sinks for nutrients, metals and sediments. This means that lakes and reservoirs can take longer to recover if contaminated.

## 2.2 ADDRESSING STORM WATER IMPACTS

The focus of this Manual is how to effectively deal with the impacts of storm water runoff through effective and comprehensive storm water management. Storm water management involves both the prevention and mitigation of storm water runoff quantity and quality impacts (as described in this chapter), and can be accomplished through a variety of methods and mechanisms.

This Manual deals with ways that developers in Northern Kentucky can effectively implement storm water management to address the impacts of new development and redevelopment, and both prevent and mitigate problems associated with storm water runoff. This is accomplished by:

- Developing land in a way that minimizes its impact on a watershed, and reduces both the amount of runoff and pollutants generated;
- Controlling storm water runoff peaks, volumes and velocities to prevent both downstream flooding and streambank channel erosion; and,
- Treating post-construction storm water runoff before it is discharged to a waterway.

# CHAPTER 3
# STORM WATER MANAGEMENT REQUIREMENTS

## 3.1 HYDROLOGIC ANALYSIS METHODS

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

- Rainfall event volumes and intensity distributions;
- Drainage area size, shape and orientation;
- Ground cover and soil type;
- Slopes of terrain and stream channel(s);
- Antecedent moisture condition;
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.);
- Watershed development potential; and,
- Characteristics of the local drainage system.

Numerous methods of rainfall-runoff computation are available on which the design of storm drainage and flood control systems may be based. Please refer to SD1's Storm Water Rules and Regulations or Boone County's Subdivision Regulations for the acceptable methods for computing storm water runoff. The engineer may use other methods with prior approval by SD1 or the City of Florence.

## 3.2 BMP SIZING REQUIREMENTS

### *3.2.1 Introduction*

In accordance with the Kentucky Pollutant Discharge Elimination System (KPDES) permit for Small Municipal Separate Storm Sewer Systems (Phase II MS4 General Permit: SD1 KPDES No. KYG200007 and City of Florence KPDES No. KYG200013), SD1 and the City of Florence have developed water quality treatment standards for new development and redevelopment projects. The following outlines the water quality treatment standard for each entity.

<u>SD1's Post-Construction Water Quality Treatment Standard in the Separate Sewer System</u>

- For **new development projects,** runoff generated from the first 0.8" of rainfall must pass through a water quality BMP. This runoff treatment standard is based on the 80th percentile precipitation event.

- For **redevelopment projects**, runoff generated from the first 0.4” of rainfall must pass through a water quality BMP. This standard is to provide greater flexibility such that redevelopment projects are not discouraged.
- Detention is as required in SD1’s Storm Water Rules and Regulations.

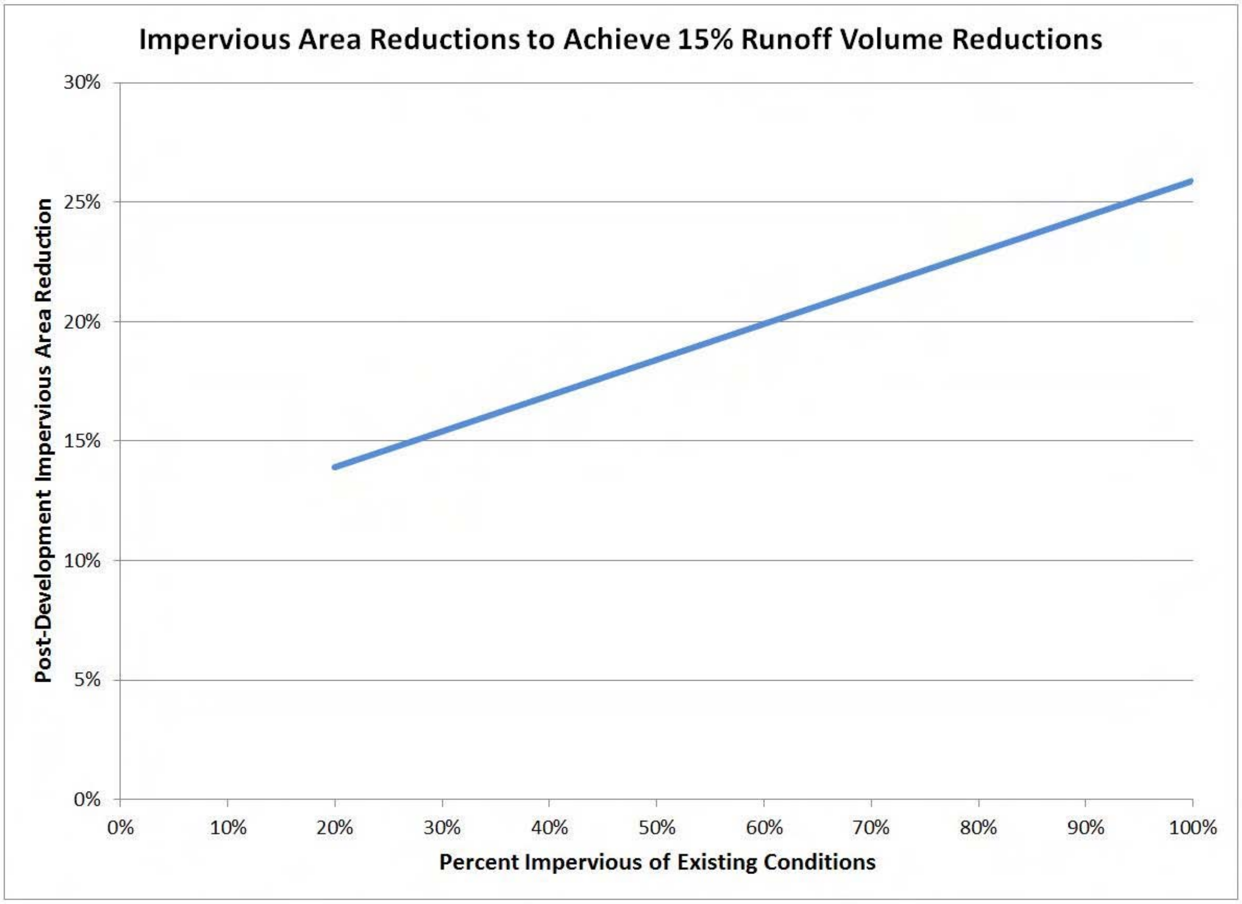
SD1’s Post-Construction Volume Reduction Standard in the Combined Sewer System

- For **new and redevelopment projects** that increase impervious area by more than 2,500 square feet, runoff generated from the first 0.8” of rainfall must pass through a volume control BMP. This standard is to minimize the impact of new development on combined sewer overflow volume. Detention is as required in SD1’s Storm Water Rules and Regulations.
- For **redevelopment projects**, that disturb more than 10,000 square feet and increase impervious area by no more than 2,500 square feet, annual runoff from the site must be reduced by 15 percent. This reduction may be accomplished by utilizing one of the following methods:

    1. Pass the runoff generated from the first 0.8 inches of rain on the entire site through an approved volume-control BMP;
    2. Reduce existing impervious area such that annual runoff from the site is reduced by 15 percent.; or,
    3. A combination of (1) and (2) above to achieve the required reduction in runoff volume.

    Detention requirements are waved for these projects as outlined in SD1’s Storm Water Rules and Regulations.

    The goal of the post-construction storm water regulation is to address storm water runoff from existing impervious area to provide positive impacts on CSO discharges and take advantage of cost-effective opportunities to reduce existing downstream flooding. The following figure highlights the relationship between existing impervious area conditions and the required reduction in impervious area during post-development conditions to achieve a 15 percent runoff volume reduction.




City of Florence's Post-Construction Water Quality Treatment Standard

- For **new development and redevelopment projects**, runoff generated from the first 0.8" of rainfall must pass through a water quality BMP. This runoff treatment standard is based on the 80th percentile precipitation event.

These BMP sizing standards are volume-based standards and are appropriate for sizing BMPs that provide their primary treatment function by storing the water quality design volume (Vwq). As such, volume-based BMPs are designed to treat a volume of runoff, which is detained for a certain period of time to allow for settling of solids and associated pollutants, as well as any biochemical treatment processes that may be provided for dissolved pollutants such as adsorption, precipitation, biodegradation, and plant uptake. Example volume-based BMPs include extended detention basins, retention basins, media bed filters, and rain gardens.

Flow based sizing standards are needed for structural BMPs that have minimal storage where their performance is related more to the peak flow rate that they are designed to treat rather than the storage capacity. As such, flow-based BMPs treat water on a continuous flow basis. Examples of flow-based BMPs include vegetated swales, filter strips, and many proprietary hydrodynamic treatment devices. These types of BMPs are more appropriately sized using a water quality design flow rate (Qwq).

While the distinction between volume-based and flow-based controls is not always clear, especially in a sequence of BMPs or BMPs that include multiple storage and flow-through treatment components, this manual differentiates these BMP types for the purposes of providing simple sizing guidelines for each type of control. **Continuous hydrologic simulation modeling may be used to demonstrate an equivalent level of treatment in lieu of the simple sizing methods presented below.**

### 3.2.2 Simple Sizing Method for Volume Based Controls

The water quality design volume used for sizing volume-based treatment BMPs may be computed using the Simple Method (Schueler, 1987). This method uses a volumetric runoff coefficient:

$$R_v = 0.009 \cdot \%IMP + 0.05 \tag{3-1}$$

Where:

$R_v$ = the volumetric runoff coefficient (unit-less)

$\%IMP$ = the percent imperviousness of the drainage area (%)

Using the design storm volume summarized above, the water quality design volume may be computed using a modified form of the rational formula:

$$V_{wq} = 3630 \cdot R_v \cdot P \cdot A \tag{3-2}$$

Where:

$V_{wq}$ = the water quality design volume ($ft^3$)

$R_v$ = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage area (see Equation 3-1 above).

P = the rainfall depth of the storm (in) [For SD1: use 0.8 for new development in the separate system, 0.4 for redevelopment in the separate system, or use 0.8 for new development and redevelopment in the combined system; for City of Florence use 0.8 for both new development and redevelopment]

A = the BMP drainage area (acres)

The water quality design volume should be used to initially size the BMP using the design criteria provided in the individual BMP fact sheets. Additional storage capacity must be provided if the BMP is designed to attenuate peak flows.

<u>Note about Drawdown Time</u>

Drawdown time is the time required to drain a volume-based BMP that has reached its design capacity, usually expressed in hours. Drawdown time is important because it is the time required to fully replenish the storage capacity, which affects the capture efficiency of the next storm, and is a surrogate for residence time, which affects treatment. Estimates for design drawdown time vary, and ideally would be determined based on site-specific information on the size, shape, and density or settling velocity of suspended particulates in the runoff. This information is generally not available and estimates of appropriate ranges for settling time have relied on settling column test information reported in literature.

An important source of drawdown time information is settling column tests conducted by Grizzard et. al. (1986) as part of the Nationwide Urban Runoff Program (NURP). Grizzard found that settling times of 48 hours resulted in removals of 80% to 90% of total suspended solids (TSS). Rapid initial removal was also observed in storm water samples with medium (100 to 215 mg/L) and high (721 mg/L) initial TSS concentrations. For example, at settling times of 24 hours, the 80% to 90% removals were already achieved in samples with medium and high initial TSS, whereas only 50% to 60% removal was achieved in those with low initial TSS.

Given the data provided above, a drawdown time of 36 to 48 hours is recommended for sizing outlet structures for volume-based BMPs that depend on settling as the primary treatment. For volume-based BMPs, such as bioretention and media filters, which depend on filtration as the primary treatment mechanism, the drawdown time for the entire system (ponded water plus the filtration media pore water) should be less than 48 hours (i.e., there is no minimum drawdown time for volume-based BMPs that include filtration as the primary treatment mechanism). The upper limit of the drawdown time is consistent with the recommendation of various vector control agencies that structures be designed to drain in less than 72 hours to minimize mosquito breeding opportunities.

### *3.2.3 Simple Sizing Method for Flow-Based Controls*

The water quality design flow rate for a flow-based BMP may be selected such that it treats an equivalent proportion of the long-term runoff volume as a volume-based BMP would. In order to use this approach, continuous runoff modeling techniques must be performed. A spreadsheet can be used to statistically analyze the long time series of runoff predicted by the continuous model for a project site to determine the flow rate associated with treating the volume of runoff determined using the volumetric sizing criteria discussed above.

An alternative simple approach is to select a design storm intensity and use the rational formula to compute the design flow rate. The design storm intensity may be based on the 80th percentile rainfall intensity. However, if hourly rainfall data are used to compute this value, the design intensity will be an under-prediction of the 80th percentile computed from shorter duration intensities. For example, during a one hour period peak rainfall, intensities may only occur for a few minutes and these peaks would be smoothed by the hourly averaging period. Therefore, a conservative approach for selecting a design storm intensity is to use twice the 80th percentile rainfall intensity from hourly historical rainfall data.

The 80th percentile hourly rainfall intensity measured at the Cincinnati-Northern Kentucky Airport is approximately 0.08 in/hr (Strecker and Rathfelder, 2008). Therefore, doubling this intensity gives a **0.16 in/hr** design storm intensity, which can be converted to a design flow rate using the rational formula:

$$Q_{wq} = R_v \cdot i \cdot A \tag{3-3}$$

Where:

| | |
|---|---|
| Qwq | = the water quality design flow rate (cfs) |
| Rv | = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage area |
| i | = rainfall intensity (in/hr) [use 0.16 in/hr] |
| A | = the BMP drainage area (acres) |

Note that 1 acre-in/hr = 1.0083 cfs; this conversion factor can be used with Equation 3-3, but is not necessary as the uncertainty for the other parameters is generally well above 0.8%.

The water quality design flow rate should be used to initially size the BMP using the design criteria provided in the individual BMP fact sheets. Additional flow capacity must be provided if the BMP is designed to convey flood flows.

### *3.2.4 References*

Grizzard T.J., C.W. Randall, B.L. Weand, and K.L. Ellis, 1986. Effectiveness of Extended Detention Ponds, in Urban Runoff Quality – Impact and Quality Enhancement Technology: pp. 323-337.

Schueler, T., 1987. "Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs," Publication No. 87703, Metropolitan Washington Council of Governments, Washington, DC.

Strecker, E. and K. Rathfelder, 2008. Water Quality BMP Sizing Evaluation for Northern Kentucky Sanitation District No. 1. Memo from Geosyntec Consultants to Kentucky Sanitation District No. 1, Fort Wright, KY, December 8.

# CHAPTER 4
# STORM WATER BETTER SITE DESIGN

CITY OF FLORENCE KENTUCKY

## 4.1 OVERVIEW

### *4.1.1 Introduction*

As discussed in *Chapter 2: The Need for Storm Water Management*, land development has the potential to impact the physical, chemical, and biological conditions of Northern Kentucky's waterways and water resources. One way of attempting to minimize these impacts is through storm water better site design practices. Development projects can be designed to reduce their impact on watersheds when careful efforts are made to conserve natural areas, reduce impervious cover and better integrate storm water treatment. By implementing a combination of these <u>nonstructural</u> approaches collectively known as storm water better site design practices, it is possible to reduce the amount of runoff and pollutants that are generated from a site and provide for some nonstructural on-site treatment and control of runoff. Although these site design practices are not required as part of the development process, they are encouraged to reduce the impacts of storm water runoff on local water resources. The goals of better site design include:

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

Better site design for storm water management includes a number of site design techniques such as preserving natural features and resources, effectively laying out the site elements to reduce impact, reducing the amount of impervious surfaces, and utilizing natural features on the site for storm water management. Many of the better site design concepts may reduce the cost of infrastructure while maintaining or even increasing the value of the property. Operationally, economically, and aesthetically, the use of better site design practices offers significant benefits over treating and controlling runoff downstream.

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

The use of storm water better site design can also have a number of other ancillary benefits including:

- Reduced construction costs;
- Increased property values;
- More open space for recreation;
- More pedestrian friendly neighborhoods;
- Protection of sensitive forests, wetlands and habitats; and,
- More aesthetically pleasing and naturally attractive landscape.

### *4.1.2 List of Storm Water Better Site Design Practices and Techniques*

The storm water better site design practices and techniques covered in this Chapter are grouped into four categories and are listed below:

- Conservation of Natural Features and Resources;
- Lower Impact Site Design Techniques;
- Reduction of Impervious Cover; and,
- Utilization of Natural Features for Storm Water Management.

More detail on each site design practice is provided in the Storm Water Better Site Design Practice Summary Sheets in subsection 4.2. These summaries provide the key benefits of each practice, examples and details on how to apply them in site design.

### *4.1.3 Using Storm Water Better Site Design Practices*

Site design should be done in unison with the design and layout of storm water infrastructure in attaining storm water management goals. Figure 4.1-1 illustrates the storm water better site design process that utilizes the four better site design categories.


Figure 4.1-1 Storm Water Better Site Design Process

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

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

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

## 4.2 BETTER SITE DESIGN PRACTICES

### *4.2.1 Conservation of Natural Features and Resources*

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

- Areas of undisturbed vegetation;

- Floodplains and riparian areas;
- Steep slopes;
- Natural drainage pathways;
- Intermittent and perennial streams;
- Wetlands;
- Aquifers and recharge areas;
- Soils;
- Shallow bedrock or high water table; and,
- Other natural features or critical areas.

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

- Preserve Undisturbed Natural Areas;
- Preserve Riparian Buffers; and,
- Avoid Floodplains.

Delineation of natural features is typically done through a comprehensive site analysis and inventory before any site layout design is performed. From a site analysis, a concept plan can be prepared that provides for the conservation and protection of natural features. Figure 4.2-1 shows an example of the delineation of natural features on a base map of a development parcel.

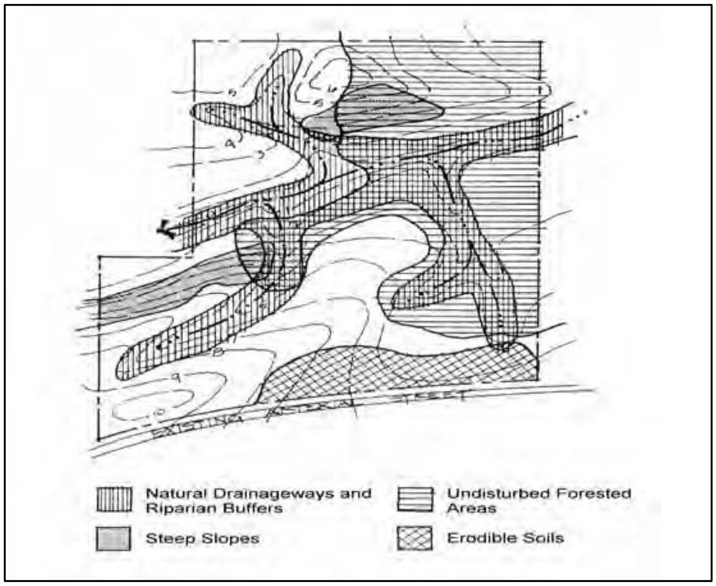



Figure 4.2-1 Example of Natural Feature Delineation
(Source: MPCA, 1989)

## 4.2.1.1 Better Site Design Practice #1

# PRESERVE UNDISTURBED NATURAL AREAS

Conservation of Natural Features and Resources

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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Conserving undisturbed natural areas helps to preserve a portion of the site's natural predevelopment hydrology<br>• Can be used as nonstructural storm water filtering and infiltration zones<br>• Helps to preserve the site's natural character and aesthetic features<br>• May increase the value of the developed property | ☑ Delineate natural areas before performing site layout and design<br>☑ Ensure that conservation areas and native vegetation are protected in an undisturbed state throughout construction and occupancy |

**Discussion**

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

Natural conservation areas are typically identified through a site analysis using maps and aerial/satellite photography, or by conducting a site visit. These areas should be delineated before any site design, clearing or construction begins. When done before the concept plan phase, the planned conservation areas can be used to guide the layout of the site. Figure 4.2-2 shows a site map with undisturbed natural areas delineated.

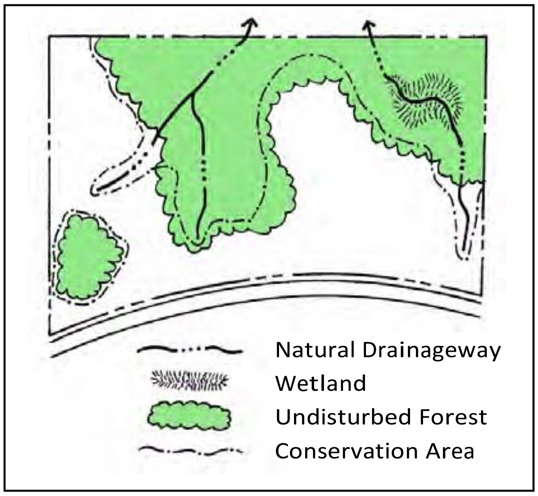



Figure 4.2-2 Delineation of Natural Conservation Areas

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

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

## PRESERVE RIPARIAN BUFFERS



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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Riparian buffers can be used as nonstructural storm water filtering and infiltration zones<br>• Keeps structures out of the floodplain and provides a right-of-way for large flood events<br>• Helps to preserve riparian ecosystems and habitats | ☑ Delineate and preserve naturally vegetated riparian buffers<br>☑ Ensure that buffers and native vegetation are protected throughout construction and occupancy |

**Discussion**

A riparian buffer is a special type of natural conservation area along a stream or wetland where development is restricted or prohibited. The primary function of buffers is to protect and physically separate a stream, lake or wetland from future disturbance or encroachment. If properly designed, a buffer can provide storm water management functions, can act as a right-of-way during floods, and can sustain the integrity of stream ecosystems and habitats. An example of a riparian stream buffer is shown in Figure 4.2-3.

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

Figure 4.2-3 Riparian Stream Buffer

Ideally, riparian buffers should be sized to include the 100-year floodplain as well as steep banks and freshwater wetlands. The buffer depth needed to perform properly will depend on the size of the stream and the surrounding conditions. Three distinct zones exist within buffer areas; these zones are shown in Figure 4.2-4. The function, vegetative target and allowable uses vary by zone as described in Table 4.2-1.

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

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

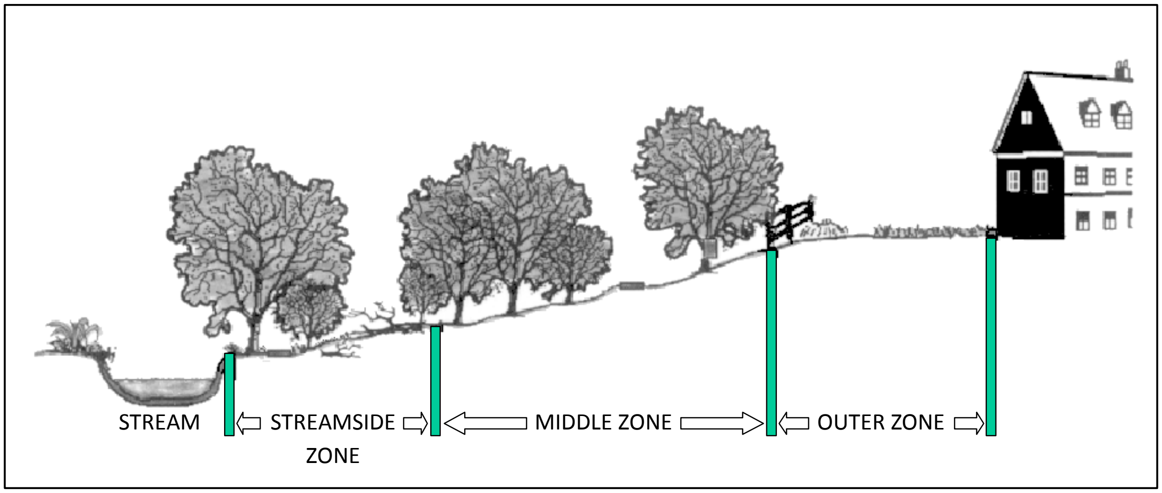



Figure 4.2-4 Three-Zone Stream Buffer System

Table 4.2-1 Riparian Buffer Management Zones

| | STREAMSIDE ZONE | MIDDLE ZONE | OUTER ZONE |
|---|---|---|---|
| Width | Minimum 25 feet plus wetlands and critical habitat | Variable depending on stream order, slope, and 100-year floodplain (min. 25 ft) | 25-foot minimum setback from structures |
| Vegetative Target | Undisturbed mature forest. Reforest if necessary. | Managed forest, some clearing allowed. | Forest encouraged, but usually turfgrass |
| Allowable Uses | Very Restricted<br>e.g., flood control, utility easements, footpaths | Restricted<br>e.g., some recreational uses, some storm water controls, bike paths | Unrestricted<br>e.g., residential uses including lawn, garden, most storm water controls |

# AVOID FLOODPLAINS



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

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

**Discussion**

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

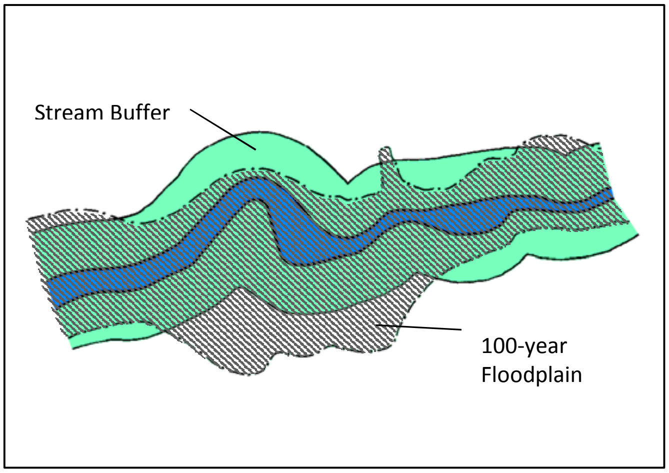



Figure 4.2-5 Floodplain Boundaries in Relation to a Riparian Buffer

As such, floodplain areas should be avoided on a development site. Ideally, the entire 100-year full-buildout floodplain should be avoided for clearing or building activities, and should be preserved in a natural undisturbed state where possible. Floodplain protection is complementary to riparian buffer preservation. Both of these better site design practices preserve stream corridors in a natural state and allow for the protection of vegetation and habitat. Depending on the site topography, 100-year floodplain boundaries may lie inside or outside of a preserved riparian buffer corridor, as shown in Figure 4.2-5.

Developers and builders should also ensure that their site design comply will any other relevant local floodplain and FEMA requirements.

# AVOID STEEP SLOPES



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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Preserving steep slopes helps to prevent soil erosion and degradation of storm water runoff<br>• Steep slopes can be kept in an undisturbed natural condition to help stabilize hillsides and soils<br>• Building on flatter areas will reduce the need for cut-and-fill and grading | ☑ Avoid development on steep slope areas, especially those with a grade of 20% or greater<br>☑ Minimize grading and flattening of hills and ridges |

**Discussion**

Developing on steep slope areas has the potential to cause excessive soil erosion and storm water runoff during and after construction. Local studies have found that soil erosion is significantly increased on slopes of 20% or greater. In addition, the nature of steep slopes means that greater areas of soil and land area are disturbed to locate facilities on them compared to flatter slopes as demonstrated in Figure 4.2-6.

Therefore, development on slopes with a grade of 20% or greater should be avoided if possible to limit soil loss, erosion, excessive storm water runoff, and the degradation of surface water. Excessive grading should be avoided on all slopes, as should the flattening of hills and ridges. Steep slopes should be kept in an undisturbed natural condition to help stabilize hillsides and soils.

On slopes greater than 20%, no development, regrading, or stripping of vegetation should be considered unless the disturbance is for roadway crossings or utility construction and it can be demonstrated that the roadway or utility improvements are absolutely necessary in the sloped area.

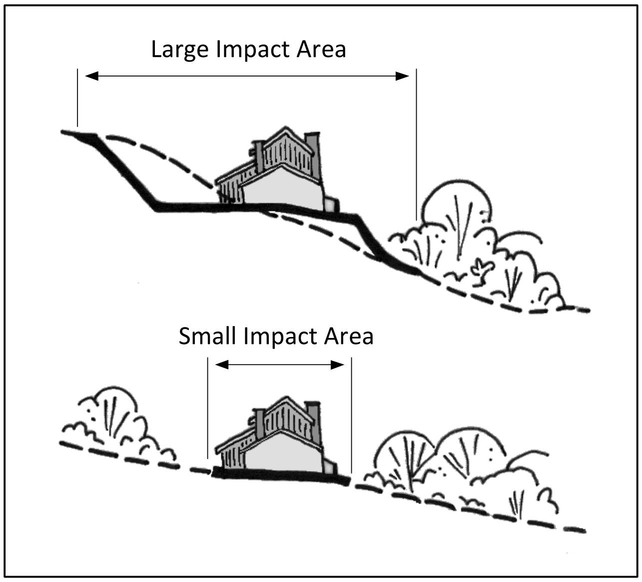



Figure 4.2-6 Flattening Steep Slopes for Building Sites Uses More Land Area Than Building on Flatter Slopes
(Source: MPCA, 1989)

# MINIMIZE SITING ON POROUS OR ERODIBLE SOILS



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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Areas with highly permeable soils can be used as nonstructural storm water infiltration zones.<br>• Avoiding high erodible or unstable soils can prevent erosion and sedimentation problems and water quality degradation | ☑ Use soil surveys to determine site soil types<br>☑ Leave areas of porous or highly erodible soils as undisturbed conservation areas |

**Discussion**

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

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

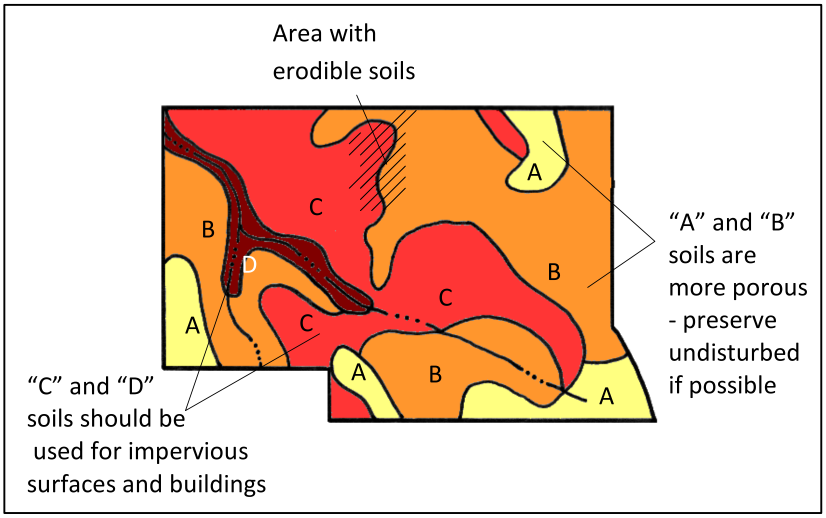



Figure 4.2-7 Soil Mapping Information Used to Guide Development

Soils on a development site should be mapped in order to preserve areas with porous soils, and to identify those areas with unstable or erodible soils as shown in Figure 4.2-7. Soil surveys can provide a considerable amount of information relating to all relevant aspects of soils. General soil types should be delineated on concept site plans to guide site layout and the placement of buildings and impervious surfaces.

### *4.2.2 Lower Impact Site Design Techniques*

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

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

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

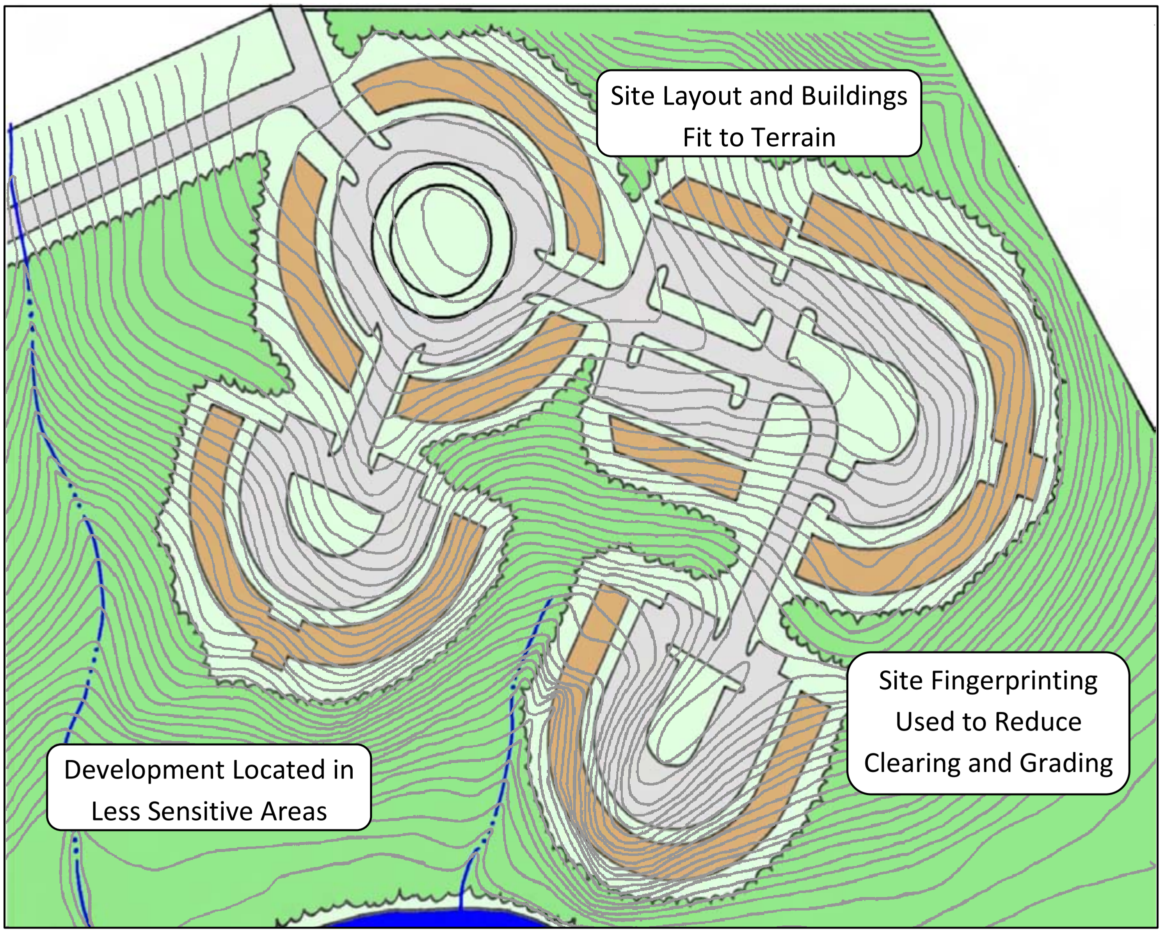



Figure 4.2-8 Development Design Utilizing Several Lower Impact Site Design Techniques

# FIT DESIGN TO THE TERRAIN



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

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

**Discussion**

All site layouts should be designed to conform with or "fit" the natural landforms and topography of a site. This helps to preserve the natural hydrology and drainageways on the site, as well as reduces the need for grading and disturbance of vegetation and soils. Figure 4.2-9 illustrates the placement of roads and homes in a residential development.

Roadway patterns on a site should be chosen to provide access schemes which match the terrain. In rolling or hilly terrain, streets should be designed to follow natural contours to reduce clearing and grading. Street hierarchies with local streets branching from collectors in short loops and cul-de-sacs along ridgelines help to prevent the crossing of streams and drainageways as shown in Figure 4.2-10. In flatter areas, a traditional grid pattern of streets or "fluid" grids which bend and may be interrupted by natural drainageways may be more appropriate (see Figure 4.2-11). In either case, buildings and impervious surfaces should be kept off of steep slopes, away from natural drainageways, and out of floodplains and other lower lying areas. In addition, the major axis of buildings should be oriented parallel to existing contours.

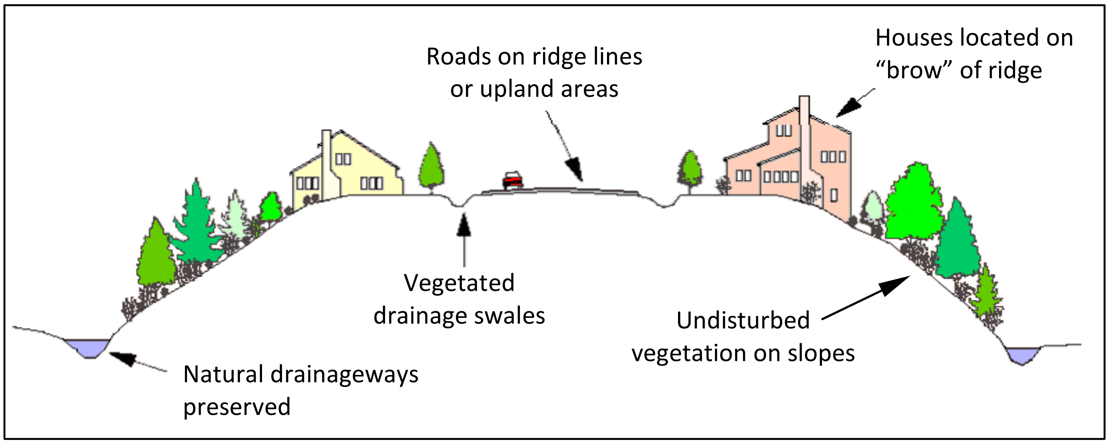



Figure 4.2-9 Preserving the Natural Topography of the Site
(Adapted from Sykes, 1989)

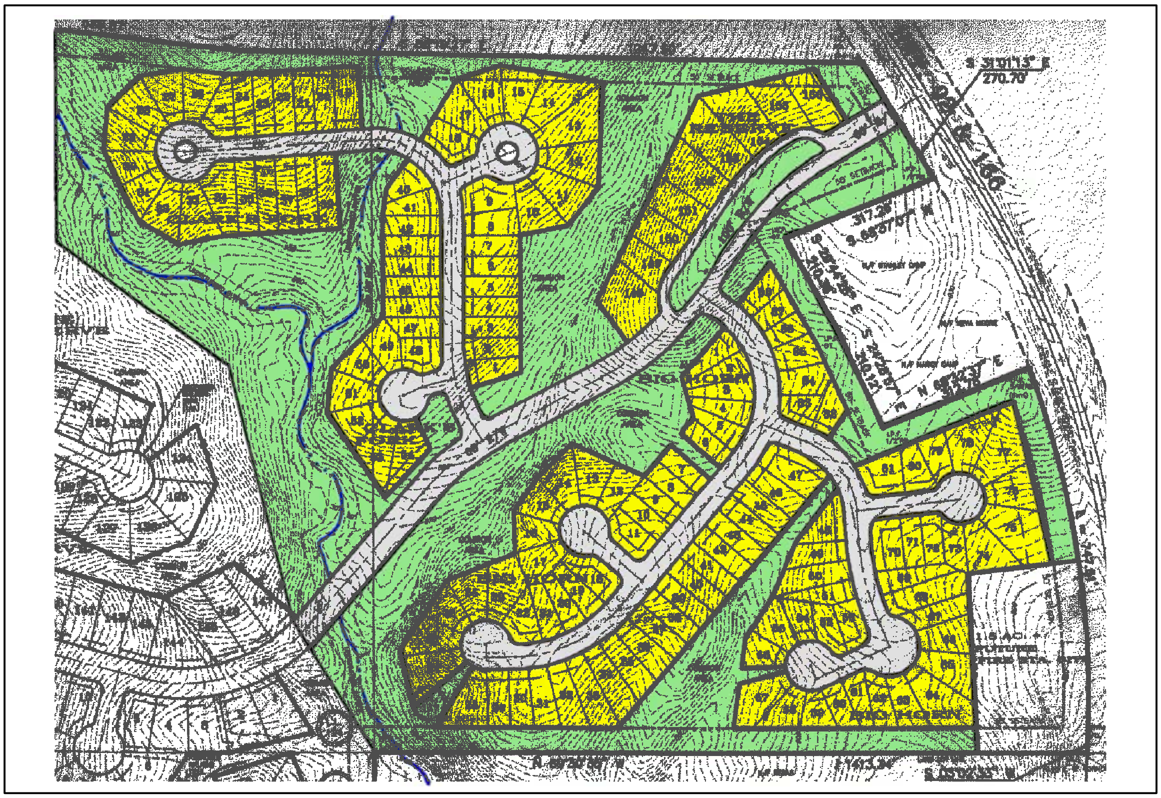


Figure 4.2-10 Subdivision Design for Hilly or Steep Terrain Utilizes Branching Streets From Collectors that Preserves Natural Drainageways and Stream Corridors

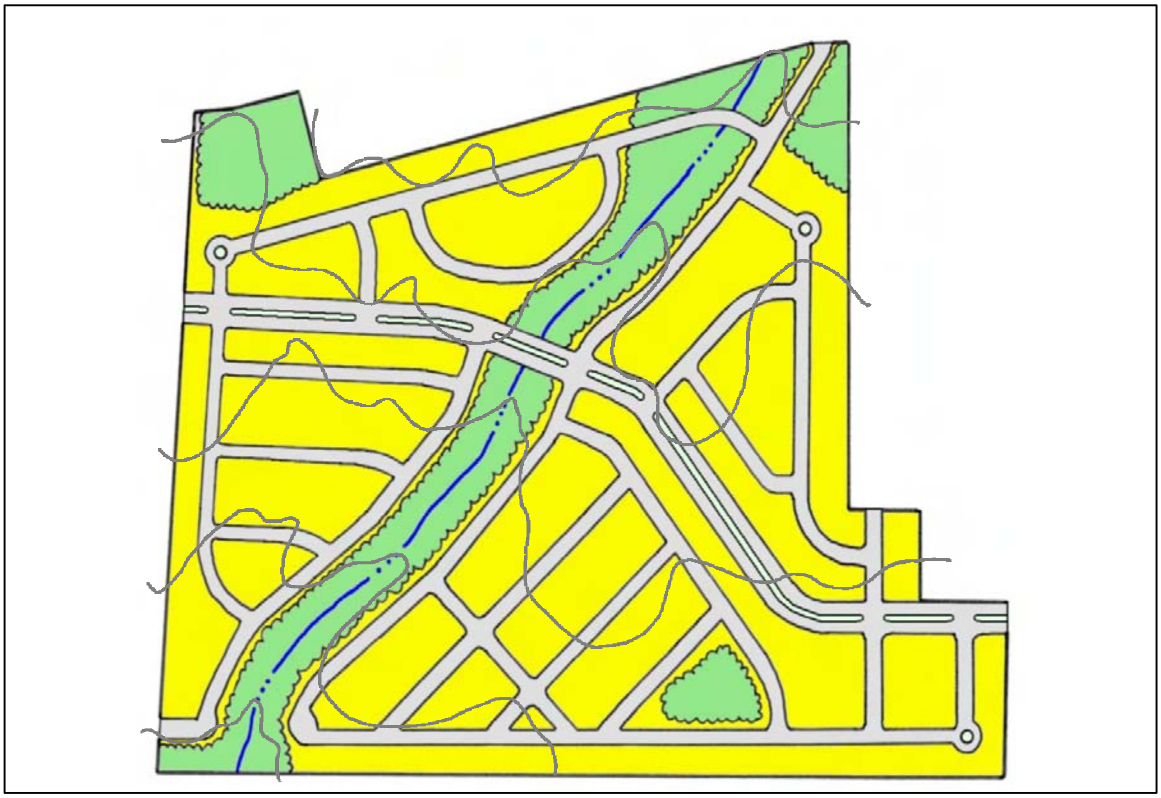


Figure 4.2-11 Subdivision Design for Flat Terrain Uses a Fluid Grid Layout that is Interrupted by the Stream Corridor

4.2.2.2 Better Site Design Practice #7

## LOCATE DEVELOPMENT IN LESS SENSITIVE AREAS



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

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

**Discussion**

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

- Locate buildings and impervious surfaces away from stream corridors, wetlands and natural drainageways. Use buffers to preserve and protect riparian areas and corridors.
- Areas of the site with porous soils should be left in an undisturbed condition and/or used as storm water runoff infiltration zones. Buildings and impervious surfaces should be located in areas with less permeable soils.
- Avoid land disturbing activities or construction on areas with steep slopes or unstable soils.
- Minimize the clearing of areas with dense tree canopy or thick vegetation, and ideally preserve them as natural conservation a

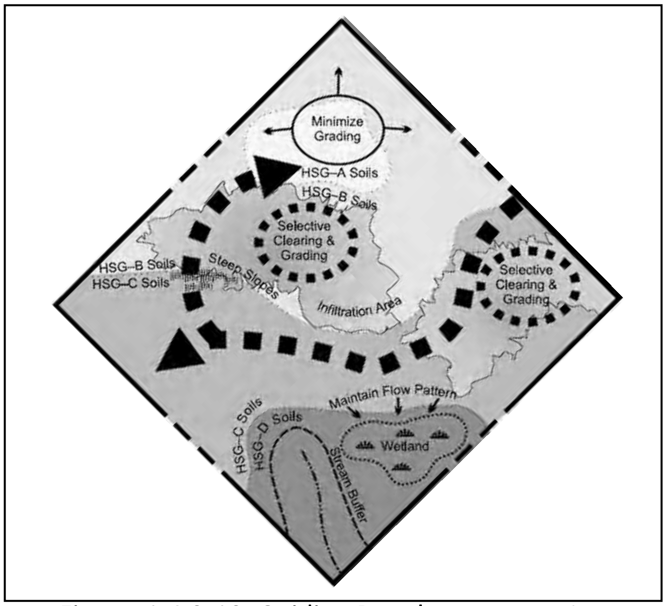



Figure 1.4.2-12 Guiding Development to Less Sensitive Areas of a Site

(Source: Prince George's County, MD, 1999)

- Ensure that natural drainageways and flow paths are preserved, where possible.
- Avoid the filling or grading of natural depressions and ponding areas.

Figure 4.2-12 shows a development site where the natural features have been mapped in order to delineate the hydrologically sensitive areas. Through careful site planning, sensitive areas can be set aside as natural open space areas (see Better Site Design Practice #9). In many cases, such areas can be used as buffer spaces between land uses on the site or between adjacent sites.

# REDUCE LIMITS OF CLEARING AND GRADING



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

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

**Discussion**

- Minimal disturbance methods should be used to limit the amount of clearing and grading that takes place on a development site, preserving more of the undisturbed vegetation and natural hydrology of a site. These methods include: Establishing a limit of disturbance (LOD) based on maximum disturbance zone radii/lengths.
- These maximum distances should reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved.
- Using site "footprinting" which maps all of the limits of disturbance to identify the smallest possible land area on a site which requires clearing or land disturbance. Examples of site footprinting are illustrated in Figures 4.2-13 and 4.2-14.
- Fitting the site design to the terrain.
- Using special procedures and equipment which reduce land disturbance.

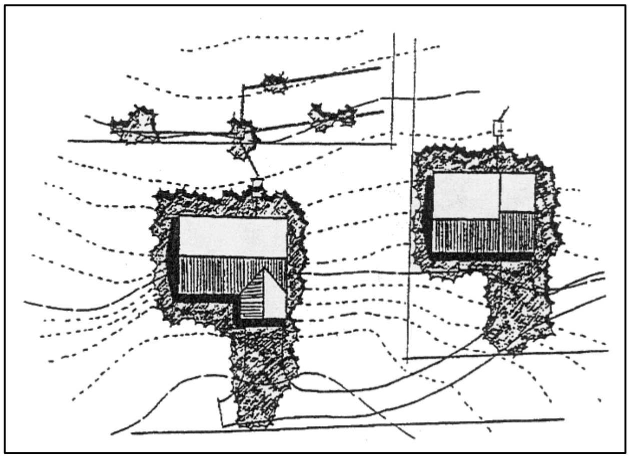


Figure 4.2-13 Establishing Limits of Clearing
(Source: DDBREC, 1997)

Figure 4.2-14 Example of Site Footprinting

## UTILIZE OPEN SPACE DEVELOPMENT



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

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

**Discussion**

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

Open space developments have many benefits compared with conventional commercial developments or residential subdivisions: they can reduce impervious cover, storm water pollution, construction costs, and the need for grading and landscaping, while providing for the conservation of natural areas. Figures 4.2-15 and 4.2-16 show examples of open space developments.

Along with reduced imperviousness, open space designs provide a host of other environmental benefits lacking in most conventional designs. These developments reduce potential pressure to encroach on conservation and buffer areas because enough open space is usually reserved to accommodate these protection areas. As less land is cleared during the construction process, alteration of the natural hydrology and the potential for soil erosion are also greatly diminished.

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

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

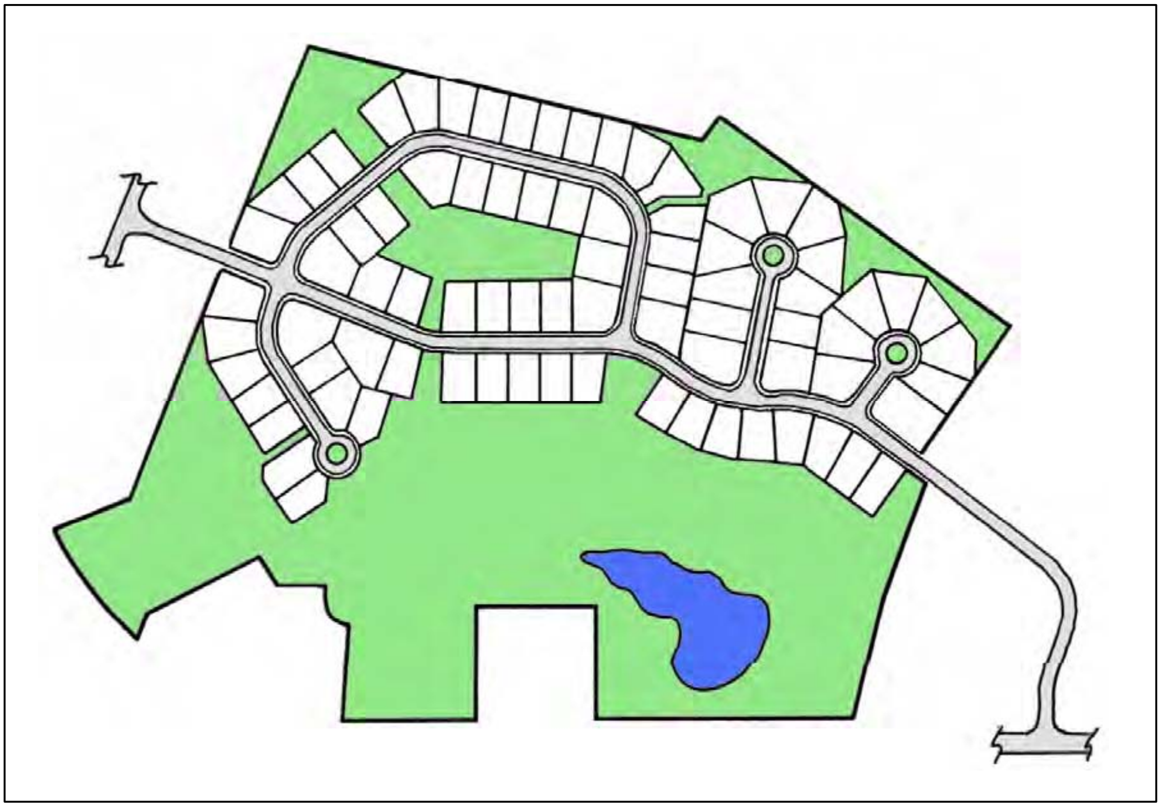


Figure 4.2-15 Open Space Subdivision Site Design Example

Figure 4.2-16 Aerial View of an Open Space Subdivision

### *4.2.3 Reduction of Impervious Cover*

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

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

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

Figure 4.2-17 Example of Reducing Impervious Cover (clockwise from upper left): (a) Cul-de-sac with Landscaped Island; (b) Narrower Residential Street; (c) Landscape Median in Roadway; and (d) "Green" Parking Lot with Landscaped Islands

### 4.2.4 Utilization of Natural Features for Storm Water Management

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

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

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

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

The following pages cover each practice in more detail.

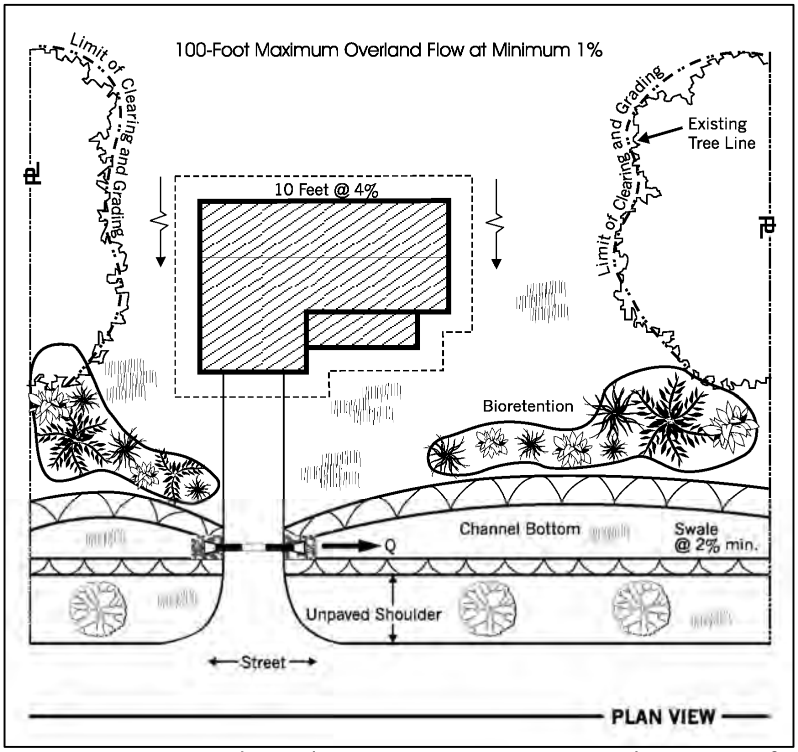



Figure 4.2-18 Residential Site Design Using Natural Features for Storm Water Management
(Source: Prince George's County, MD, 1999)

4.2.4.1 Better Site Design Practice #10

## USE BUFFERS AND UNDISTURBED AREAS

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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Riparian buffers and undisturbed vegetated areas can be used to filter and infiltrate storm water runoff<br>• Natural depressions can provide inexpensive storage and detention of storm water flows | ☑ Direct runoff towards buffers and undisturbed areas using a level spreader to ensure sheet flow<br>☑ Utilize natural depressions for runoff storage |

**Discussion**

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

The objective in utilizing natural areas for storm water infiltration is to intercept runoff before it has become substantially concentrated and then distribute this flow evenly (as sheet flow) to the buffer or natural area. This can typically be accomplished using a level spreader, as seen in Figure 4.2-19. A mechanism for the bypass of higher flow events should be provided to reduce erosion or damage to a buffer or undisturbed natural area.

Carefully constructed berms can be placed around natural depressions and below undisturbed vegetated areas with porous soils to provide for additional runoff storage and/or infiltration of flows.

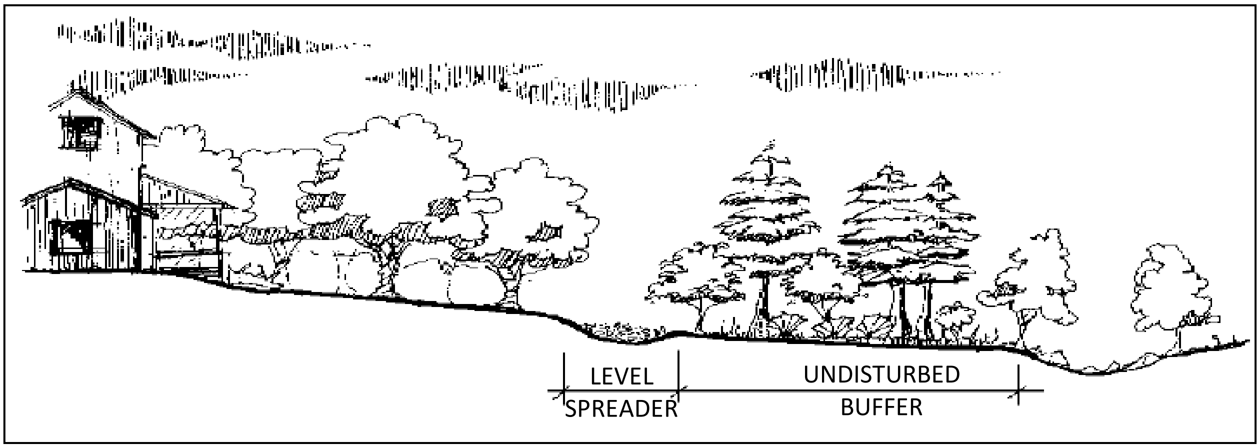



Figure 4.2-19 Use of a Level Spreader with a Riparian Buffer
(Adapted from NCDENR, 1998)

4.2.4.2 Better Site Design Practice #11

## NATURAL DRAINAGEWAYS INSTEAD OF STORM SEWER

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

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

**Discussion**

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

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

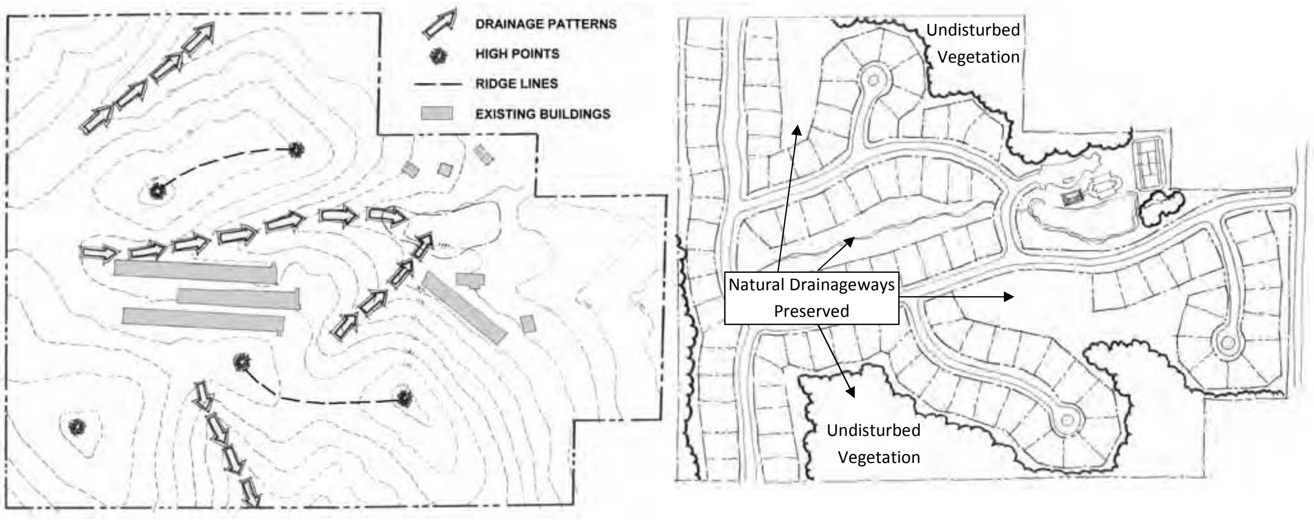



Figure 4.2-20 Example of a Subdivision Using Natural Drainageways for Storm Water Conveyance and Management

4.2.4.3 Better Site Design Practice #12

## VEGETATED SWALES INSTEAD OF CURB AND GUTTER

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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Reduces the cost of road and storm sewer construction<br>• Provides for some runoff storage and infiltration, as well as treatment of storm water | ☑ Use vegetated open channels (enhanced wet or dry swales or grass channels) in place of curb and gutter to convey and treat storm water runoff |

**Discussion**

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

Open vegetated channels along a roadway (see Figure 4.2-21) remove pollutants by allowing infiltration and filtering to occur, unlike curb and gutter systems which move water with virtually no treatment. Engineering techniques have advanced the roadside ditches of the past, which suffered from erosion, standing water and break up of the road edge. Grass channels and enhanced dry swales are two such alternatives and with proper installation under the right site conditions, they are excellent methods for treating storm water on-site. In addition, open vegetated channels can be less expensive to install than curb and gutter systems. Further design information and specifications for grass channels and enhanced swales can be found in Chapter 7.

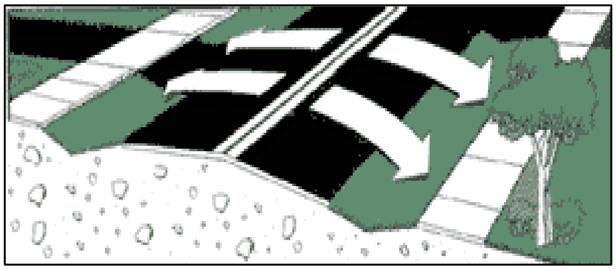


Figure 4.2-21 Using Vegetated Swales Instead of Curb and Gutter

## DRAIN RUNOFF TO PERVIOUS AREAS



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

| KEY BENEFITS | USING THIS PRACTICE |
|---|---|
| • Sending runoff to pervious vegetated areas increases overland flow time and reduces peak flows<br>• Vegetated areas can often filter and infiltrate storm water runoff | ☑ Minimize directly connected impervious areas and drain runoff as sheet flow to pervious vegetated areas |

**Discussion**

Storm water quantity and quality benefits can be achieved by routing the runoff from impervious areas to pervious areas such as lawns, landscaping, filter strips and vegetated channels. Much like the use of undisturbed buffers and natural areas (Better Site Design Practice #10), revegetated areas such as lawns and engineered filter strips and vegetated channels can act as biofilters for storm water runoff and provide for infiltration in porous (hydrologic group A and B) soils. In this way, the runoff is "disconnected" from a hydraulically efficient structural conveyance such as a curb and gutter or storm drain system.

Some of the methods for disconnecting impervious areas include:

- Designing roof drains to flow to vegetated areas;
- Directing flow from paved areas such as driveways to stabilized vegetated areas;
- Breaking up flow directions from large paved surfaces (see Figure 4.2-22); and,
- Carefully locating impervious areas and grading landscaped areas to achieve sheet flow runoff to the vegetated pervious areas.

For maximum benefit, runoff from impervious areas to vegetated areas must occur as sheet flow and vegetation must be stabilized. See Chapter 7 for more design information and specifications on filter strips and vegetated channels.

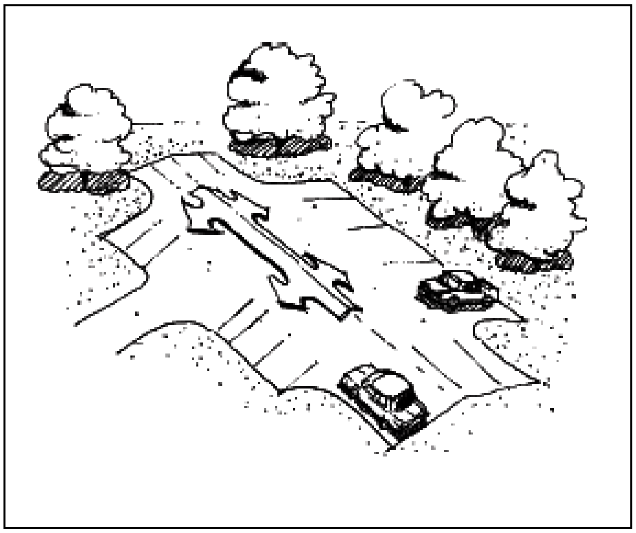


Figure 4.2-22 Design Paved Surfaces to Disperse Flow to Vegetated Areas (Source: NCDENR, 1998)

## 4.3 BETTER SITE DESIGN EXAMPLES

### *4.3.1 Residential Subdivision Example 1*

A typical residential subdivision design on a parcel is shown in Figure 4.3-1(a). The entire parcel except for the subdivision amenity area (clubhouse and tennis courts) is used for lots. The entire site is cleared and mass graded, and no attempt is made to fit the road layout to the existing topography. Because of the clearing and grading, all of the existing tree cover and vegetation and topsoil are removed dramatically altering both the natural hydrology and drainage of the site. The wide residential streets create unnecessary impervious cover and a curb and gutter system that carries storm water flows to the storm sewer system. No provision for non-structural storm water treatment is provided on the subdivision site.

A residential subdivision employing storm water better site design practices is presented in Figure 4.3-1(b). This subdivision configuration preserves a quarter of the property as undisturbed open space and vegetation. The road layout is designed to fit the topography of the parcel, following the high points and ridgelines. The natural drainage patterns of the site are preserved and are utilized to provide natural storm water treatment and conveyance. Narrower streets reduce impervious cover and grass channels provide for treatment and conveyance of roadway and driveway runoff. Landscaped islands at the ends of cul-de-sacs also reduce impervious cover and provide storm water treatment functions. When constructing and building homes, only the building envelopes of the individual lots are cleared and graded, further preserving the natural hydrology of the site.

### *4.3.2 Residential Subdivision Example 2*

Another typical residential subdivision design is shown in Figure 4.3-2(a). Most of this site is cleared and mass graded, with the exception of a small riparian buffer along the large stream at the right boundary of the property. Almost no buffer was provided along the small stream that runs through the middle of the property. In fact, areas within the 100-year floodplain were cleared and filled for home sites. As is typical in many subdivision designs, this one has wide streets for on-street parking and large cul-de-sacs.

The better site design subdivision can be seen in Figure 4.3-2(b). This subdivision layout was designed to conform to the natural terrain. The street pattern consists of a wider main thoroughfare that winds through the subdivision along the ridgeline. Narrower loop roads branch off of the main road and utilize landscaped islands. Large riparian buffers are preserved along both the small and large streams. The total undisturbed conservation area is close to one-third of the site.

### *4.3.3 Commercial Development Example*

Figure 4.3-3(a) shows a typical commercial development containing a supermarket, drugstore, smaller shops and a restaurant on an outlot. The majority of the parcel is a concentrated parking lot area. The only pervious area is a small replanted vegetation area acting as a buffer between the shopping center and adjacent land uses. Storm water quality and quantity control are provided by a wet extended detention pond in the corner of the parcel.

A better site design commercial development can be seen in Figure 4.3-3(b). Here the retail buildings are dispersed on the property, providing more of an “urban village” feel with pedestrian access between the buildings. The parking is broken up, and bioretention areas for storm water treatment are built into parking lot islands. A large bioretention area which serves as open green space is located at the main entrance to the shopping center. A larger undisturbed buffer has been preserved on the site. Because the bioretention areas and buffer provide water quality treatment, only a dry extended detention basin is needed for water quantity control.

### 4.3.4 Office Park Example

An office park with a conventional design is shown in Figure 4.3-4(a). Here the site has been graded to fit the building layout and parking area. All of the vegetated areas of this site are replanted areas.

The better site design layout, presented in Figure 4.3-4(b), preserves undisturbed vegetated buffers and open space areas on the site. Both the parking areas and buildings have been designed to fit the natural terrain of the site. In addition, a modular porous paver system is used for the overflow parking areas.

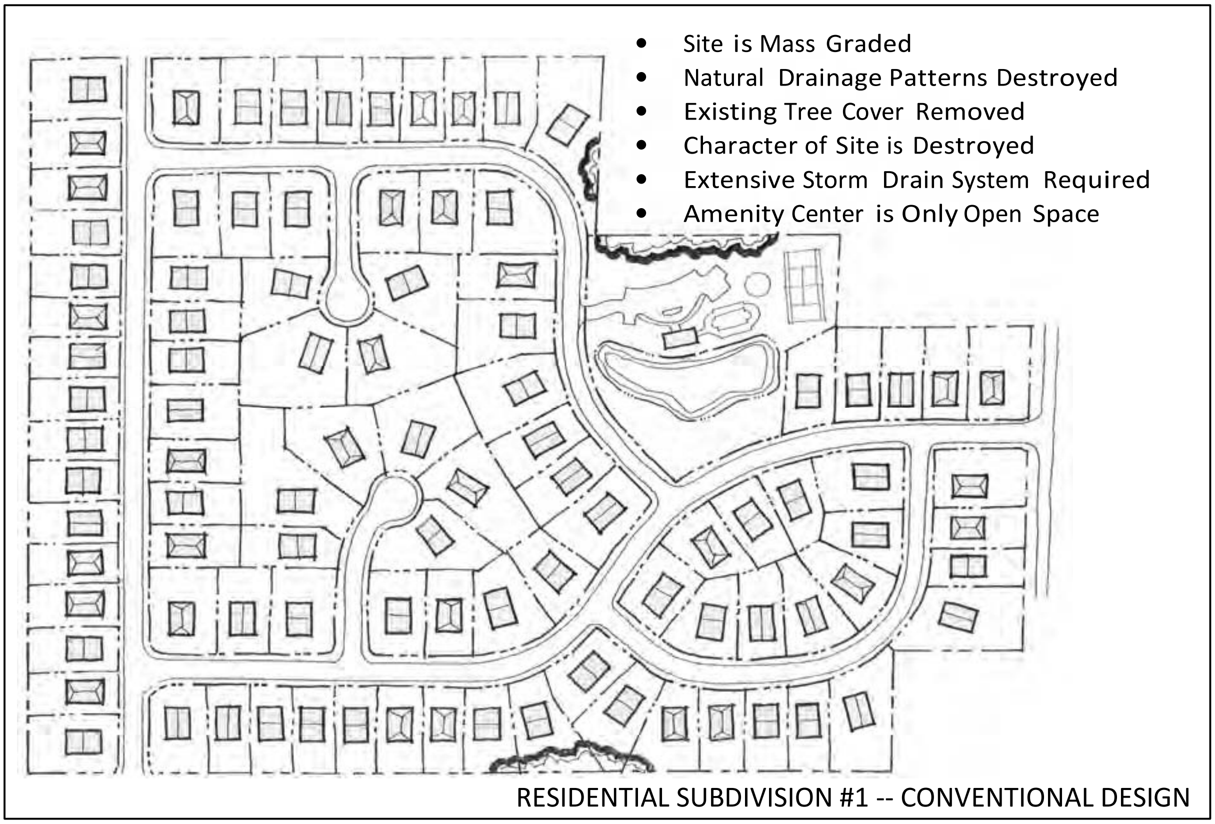
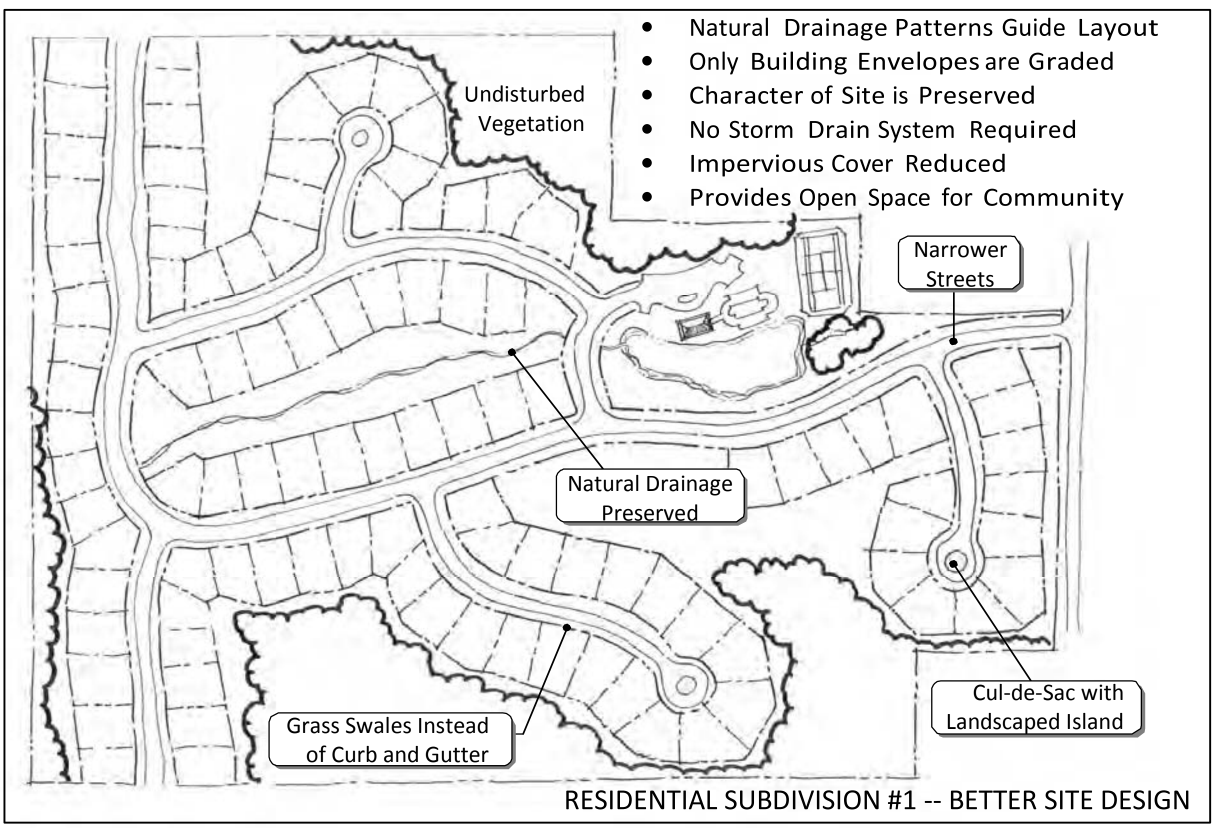



Figure 4.3-1 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



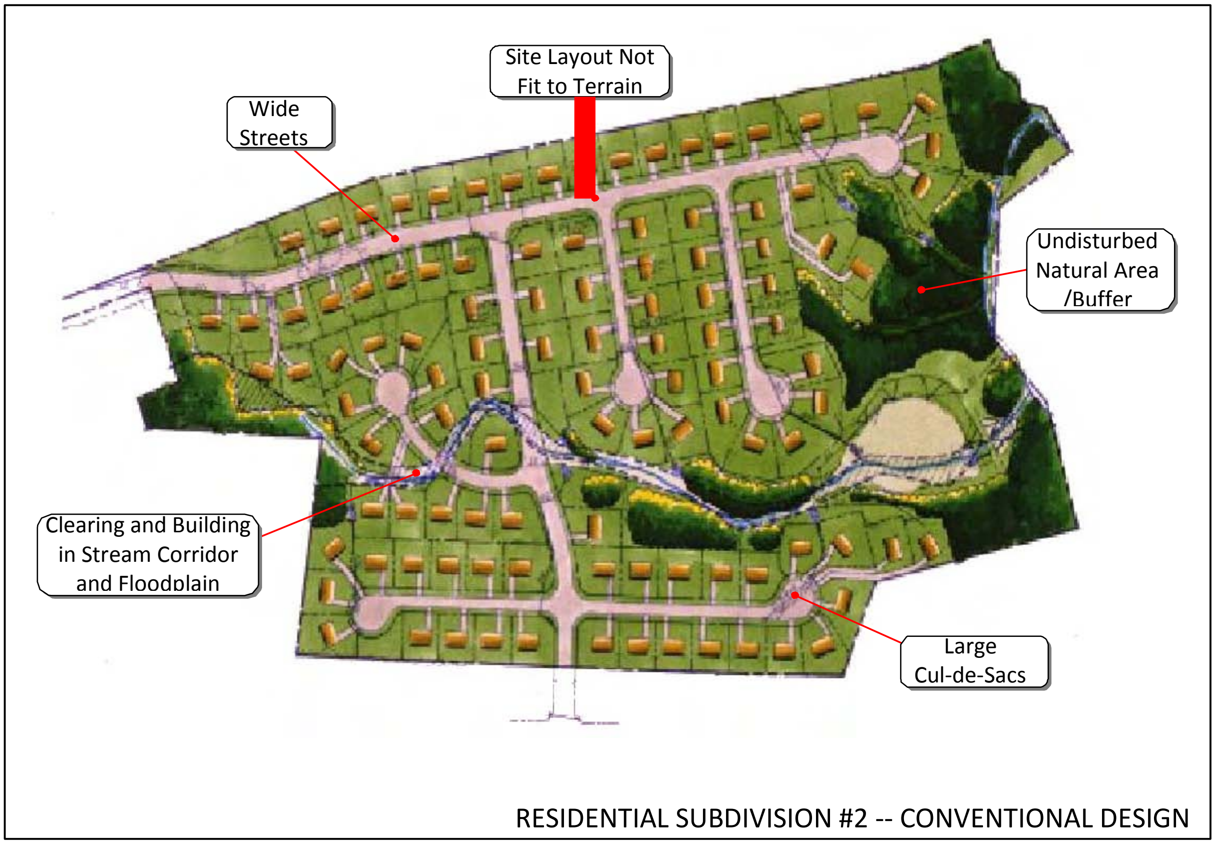
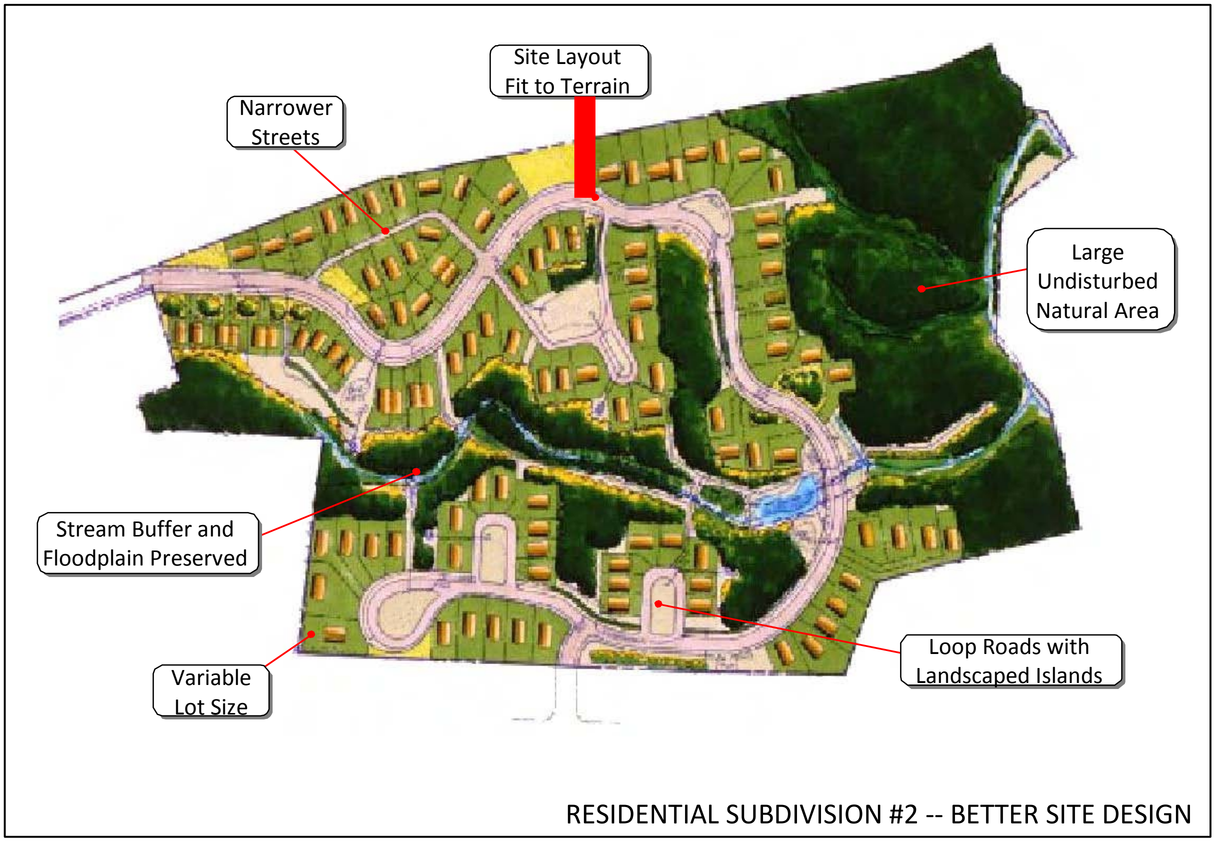



Figure 4.3-2 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



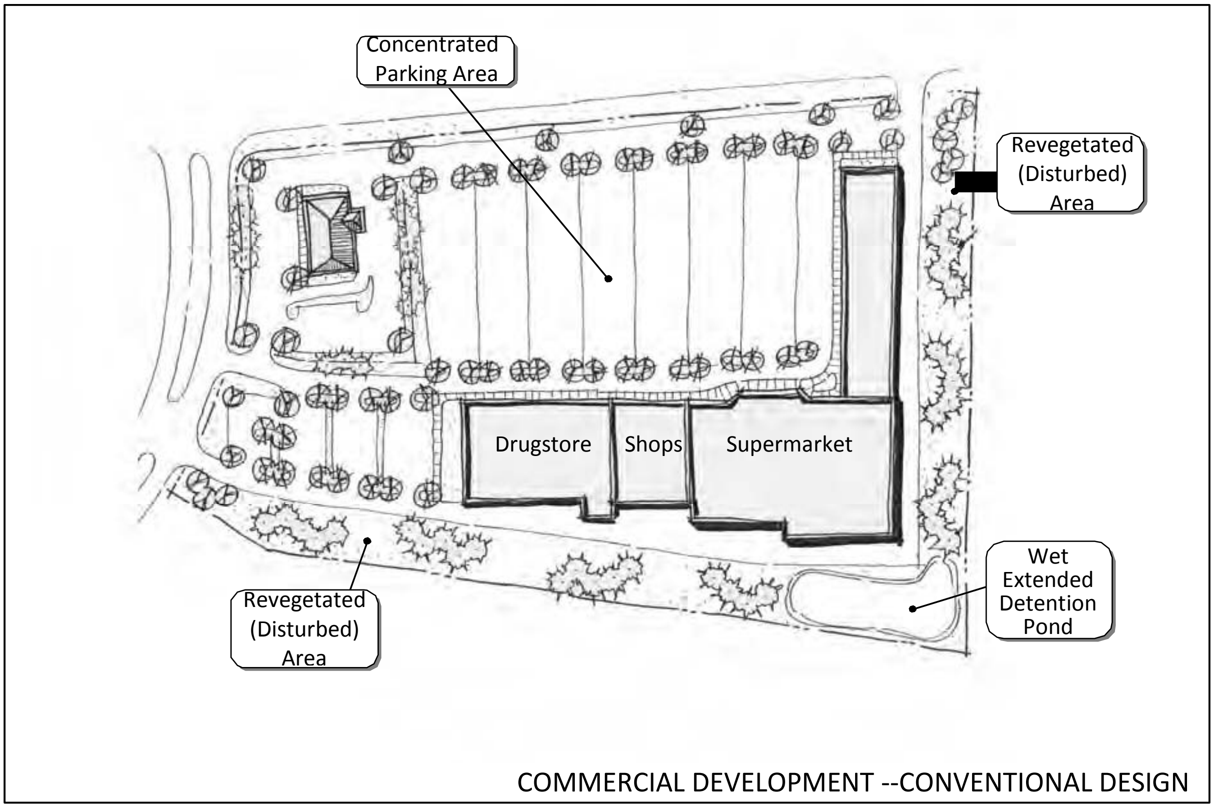
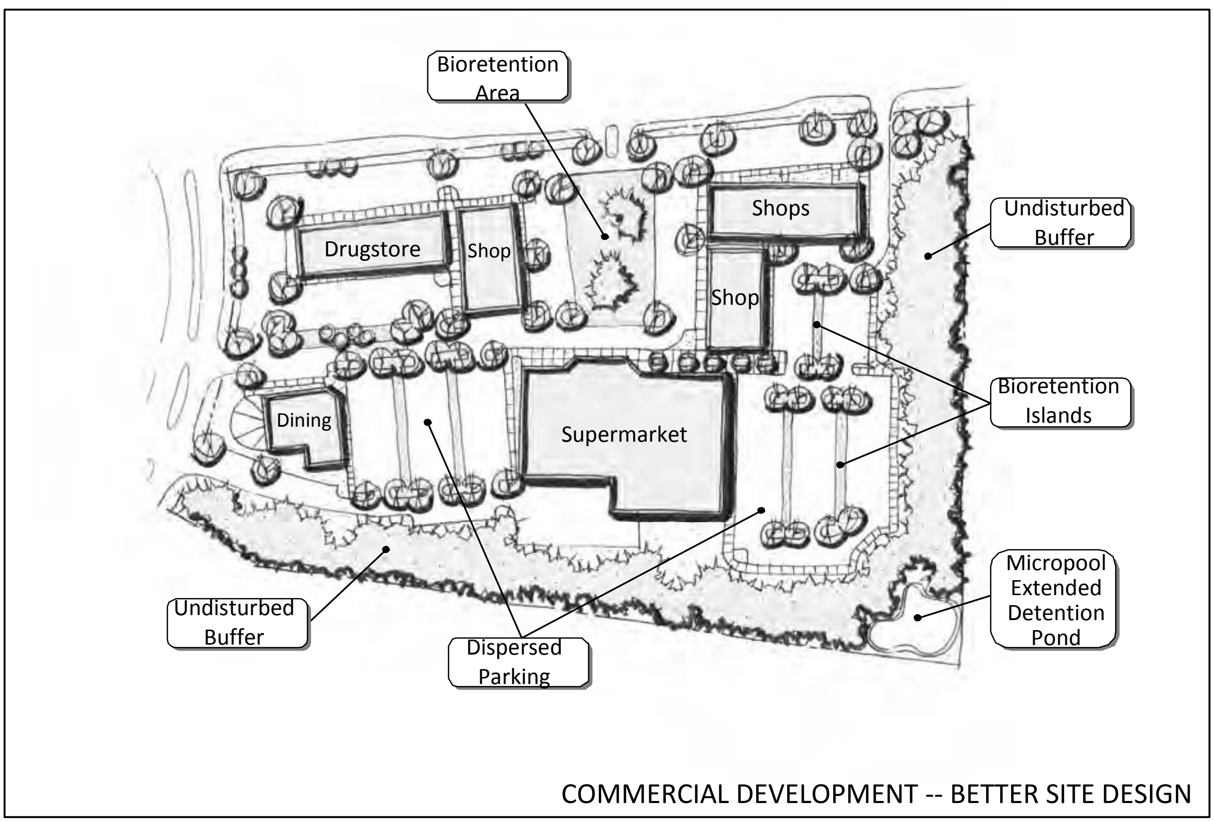



Figure 4.3-3 Comparison of a Traditional Commercial Development (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



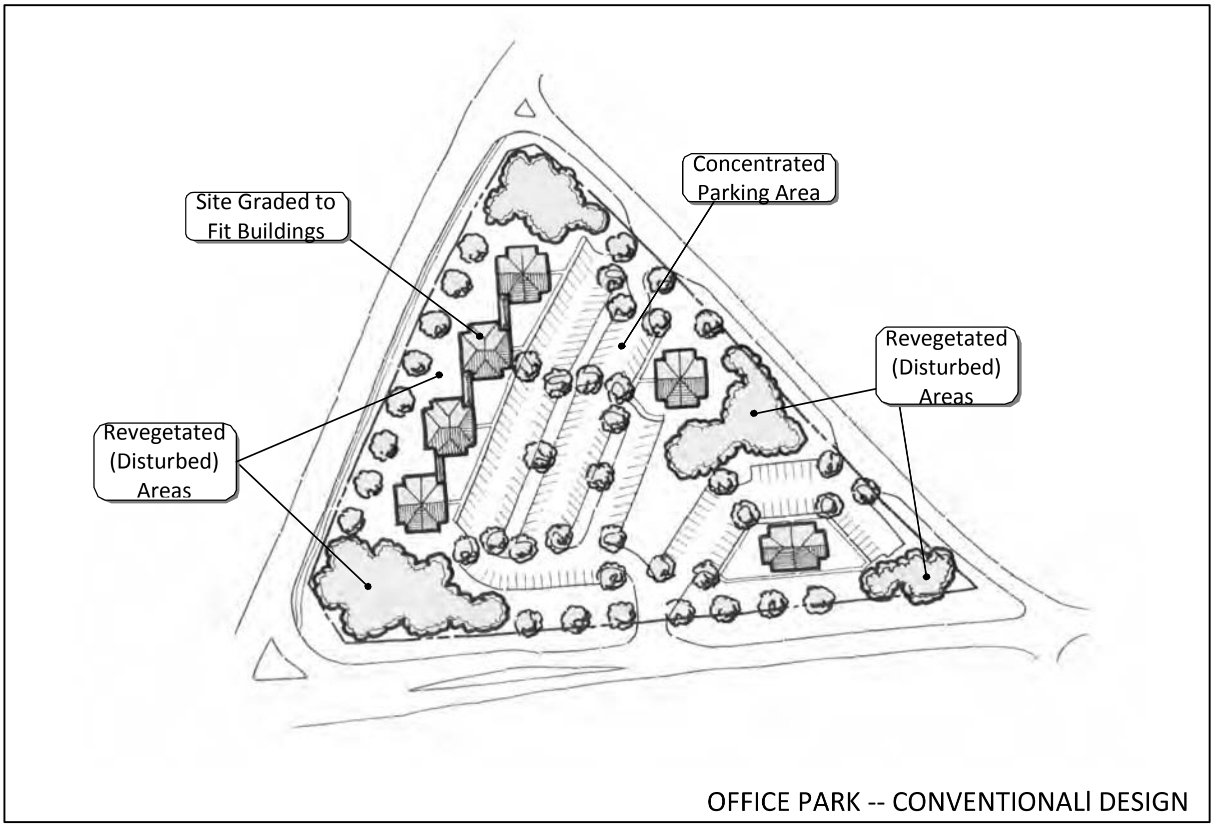
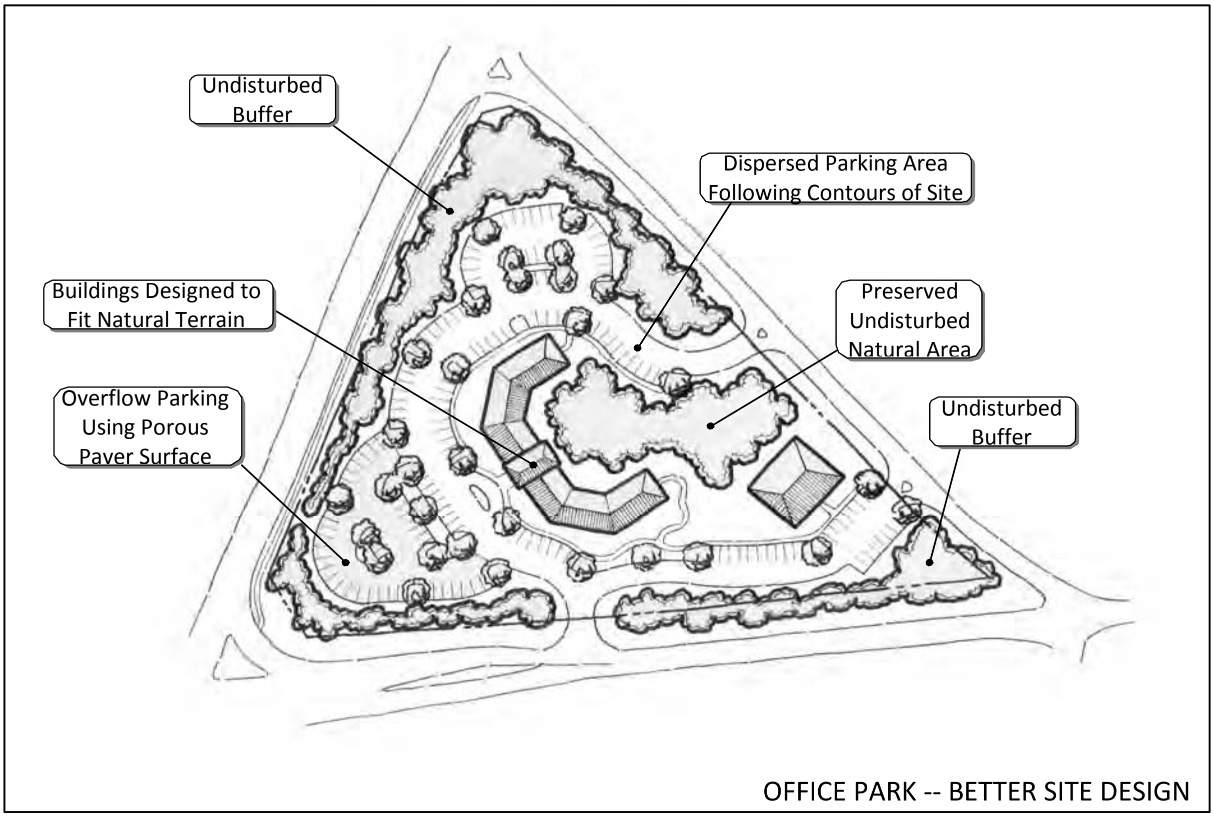



Figure 4.3-4 Comparison of a Traditional Office Park Design (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



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# CHAPTER 5
## STORM WATER SITE PLANNING

## 5.1 STORM WATER MANAGEMENT AND SITE PLANNING

### *5.1.1 Introduction*

In order to most effectively address storm water management objectives, consideration of storm water runoff needs to be fully integrated into the site planning and design process. This involves a more comprehensive approach to site planning and a thorough understanding of the physical characteristics and resources of the site. The purpose of this section is to provide a framework for including effective and environmentally sensitive storm water management into the site development process and to encourage a greater uniformity in storm water management site plan preparation.

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

- How can the storm water management system be designed to most effectively meet the storm water management minimum standards (and any additional needs or objectives)?
- What are the opportunities for utilizing alternative site design practices to minimize the need for structural storm water controls?
- What are the development site constraints that preclude the use of certain structural controls?
- What structural controls are most suitable and cost-effective for the site?

### *5.1.2 Principles of Storm Water Management Site Planning*

The following principles should be kept in mind in preparing a storm water management plan for a development site:

1. **The site design should utilize an integrated approach to deal with storm water quantity, quality and streambank (channel) protection requirements.**

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

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

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

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

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

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

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

   A structural storm water facility has the ability to be effectively and aesthetically integrated into a development site. A parking lot, soccer field or city plaza can serve as a temporary storage facility for storm water. In addition, water features such as ponds and lakes, when correctly designed and integrated into a site, can increase the aesthetic value of a development.

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

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

## 5.2 PREPARING A STORM WATER MANAGEMENT PLAN

The preparation of a storm water site plan ideally follows these steps:

(1) Review of Local Requirements;

(2) Perform Site Analysis;

(3) Prepare Storm Water Site Plan; and,

(4) Obtain Necessary Non-Local Permits.

### *5.2.1 Review of Local Requirements*

The site developer should become familiar with the local storm water management and development requirements and design criteria that apply to the site. These requirements may include:

- The minimum standards for post-construction storm water management;
- Design storm frequencies;
- Conveyance design criteria;
- Buffer criteria;
- Erosion prevention and sediment control requirements;
- Maintenance requirements; and,
- Need for physical site evaluations (infiltration tests, geotechnical evaluations, etc.).

These requirements may be found in SD1's Storm Water Rules and Regulations, Boone County's Subdivision Regulations, and/or Northern Kentucky's BMP Manual. Current land use plans, comprehensive plans, zoning ordinances, road and utility plans, and public facility plans should all be consulted to determine the need for compliance with other local and state regulatory requirements.

### *5.2.2 Perform Site Analysis and Inventory*

Using appropriate field and mapping techniques, the site engineer should review existing site conditions including, but not limited to:

- Topography;
- Drainage patterns and basins;
- Soils;
- Ground cover and vegetation;
- Existing development; and,
- Existing storm water facilities.

### *5.2.3 Prepare Storm Water Site Plan*

Based upon the review of existing conditions and site analysis, a concept layout plan may be developed for the project. Preliminary storm water management practices may be incorporated into the plan at this time. It is recommended that the following steps be considered in designing the storm water management plan:

- Preserve the natural feature conservation areas defined in the site analysis;
- Fit the development to the terrain and minimize land disturbance;
- Reduce impervious surfaces in the development; and,
- Preserve and utilize the natural drainage system wherever possible.

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

### *5.2.4 Obtain Non-Local Permits*

The developer should obtain any applicable non-local environmental permit such as Army Corps of Engineers permits (River and Harbors Act of 1899: "Section 10" - regulates the placement of any structure or work in, under, or over a "traditionally navigable water"; CWA Section 404 regulates the discharge of dredged or fill material into "waters of the U.S.") and Commonwealth of Kentucky permits (Water Quality Certification 401 Program, Floodplain Construction Program or KPDES General Permit for Stormwater Discharges Associated with Construction Activities (KYR10)) prior to or in conjunction with final plan submittal.

The following sources may provide information on applicable permits:

- Refer to SD1's Permit Guidance Document for information, instructions, and forms for both local and non-local permits that may be applicable (*note: this document is not all inclusive*). SD1's Permit Guidance Document is currently being updated, but is available upon request through SD1's website: www.sd1.org.

- Commonwealth of Kentucky Energy and Environment Cabinet *Forms Library and Related Documents* page: http://dep.ky.gov/formslibrary/Pages/default.aspx

- U.S. Army Corps of Engineers *Regulatory (Permits)* page: http://www.usace.army.mil/CECW/Pages/cecwo_reg.aspx

# CHAPTER 6
## BMP SELECTION GUIDANCE

This chapter provides a method for storm water treatment BMP selection procedures based on observed performance data and a method of pollutant removal approach for addressing Northern Kentucky's primary pollutants and parameters of concern. Additionally, water quality performance data for various storm water treatment BMP types is included in Appendix A as supplementary reference information. Together, this information is intended to assist SD1, the City of Florence and the Northern Kentucky development community in selecting and designing post-construction storm water BMPs appropriate for addressing the water quality and water quantity concerns of the region; it will also assist SD1 in successful implementation of the green infrastructure program for targeting reduction of CSOs, SSOs, stream erosion, and other activities responsible for contributing pollutants of concern to the local water bodies.

## 6.1 BMP SELECTION PROCEDURE

The recommended process for selecting storm water treatment BMPs involves a series of five steps as identified in Figure 6.1-1. Each of the five steps will be described in the following subsections. The BMP selection procedure described herein provides:

(1) A method for determining the pollutants of concern for which selected storm water treatment BMPs should target;

(2) Other factors/constraints that can influence BMP selection, such as regulatory requirements, peak flow concerns, hydrology and hydraulics, site specific constraints, regional constraints, aesthetics, cost, reliability, safety, and maintenance considerations; and

(3) Information on the method of pollutant removal for different BMP treatment system components that target the identified pollutants of concern.

Appendix A includes BMP performance information as a supplement to the information on the method of pollutant removal   This Appendix presents detailed performance information for several types of storm water treatment BMPs.

STEP 1
Define Overall Project Objectives

STEP 2
Identify Primary Pollutants of Concern From Land Use Characteristics and Known Water Quality Issues Identified on the Section 303(d) List of Impaired Receiving Waters

STEP 3
Identify Appropriate BMP to Treat the Primary Pollutants of Concern based on the Applicable Methods of Pollutant Removal

STEP 4
Identify Other Factors/Constraints That May Affect Treatment BMP Selection

STEP 5
Select Treatment BMPs Based on Primary Pollutants of Concern, Method of Pollutant Removal for Targeting Primary Pollutants of Concern, and Other Factors/Constraints

Figure 6.1-1 Storm Water Treatment BMP Selection Procedure

### *6.1.1 Step 1: Define Overall Project Objectives for Storm Water Treatment*

The design of any engineering system requires a clear definition of the problem. Without clear descriptions of the storm water issues that need to be addressed, including the desired results, it is difficult to evaluate the steps needed to select and design a practicable and cost-effective storm water treatment system. The following key concepts should be considered when defining the overall objectives of projects:

- What is the overall project scope and objectives beyond storm water treatment?
- How do the storm water management objectives relate to or conflict with other project objectives?
- What site conditions (e.g., land use types, topography, soil types, receiving waters) should be evaluated to properly define the problem?

### 6.1.2 Step 2: Identify Primary Pollutants of Concern

Primary pollutants of concern are identified based on the consideration of general land use characteristics, and known water quality issues identified on the Section 303(d) list of impaired receiving waters within Northern Kentucky (i.e., Boone, Campbell, and Kenton Counties). The recommended process for determining the primary pollutants of concern based on land use and 303(d) listed receiving waters is provided in this section. Once the primary pollutants of concern are determined, appropriate storm water treatment BMPs and/or BMP design components can be selected. Generally, pollutants of concern for the Northern Kentucky region are bacteria, nutrients and sediment.

After identifying the receiving waters for the project area, review the most current 303(d) listed surface waters for Boone, Campbell, and Kenton Counties and identify the pollutants of concern for the identified receiving waters (visit http://water.ky.gov/). Table 6.1-1 provides a general understanding, or level of occurrence, of the pollutants of concern impacting Northern Kentucky's surface waters. It is recommended that selected storm water treatment BMPs specifically address the primary pollutants of concern for this region.

Table 6.1-1 Overview of Northern Kentucky's Pollutants of Concern

| POLLUTANT CATEGORY OF CONCERN | LEVEL OF OCCURRENCE* |
|---|---|
| Bacteria<br>(Fecal coliform, *E. coli*) | High |
| Organic Enrichment<br>(Sewage) | Moderate |
| Sedimentation/Siltation/Turbidity | Moderate |
| Nutrient/Eutrophication | Moderate to High |

*Level of Occurrence based on miles of impacted stream from the 2010 303(d) list, excluding the Ohio River. Visit http://water.ky.gov/ to review the most current 303(d) listed surface waters for Boone, Campbell, and Kenton Counties.

The following describes the pollutants of concern, common sources, and common problems caused by the pollutant for pollutants currently impacting Northern Kentucky and other common issues that may impact local receiving waters in the future.

Bacteria/Pathogens

Elevated levels of human pathogen indicator bacteria are common in urban runoff. Runoff that flows over land such as urban runoff can mobilize pathogens, including bacteria and viruses. Even runoff from natural areas may contain pathogens from wildlife excrement. Sources of pathogens in urban areas include domestic animal waste, wild fauna, septic systems, leaking sanitary sewer pipes, and combined sewer overflows.

Organic Enrichment/Oxygen Demanding Substances

Dissolved oxygen is a basic requirement for a healthy aquatic ecosystem. Most fish and aquatic insects "breathe" oxygen dissolved in the water column. Oxygen concentrations in the water fluctuate under natural conditions, but severe depletion usually results from sources that introduce large quantities of biodegradable organic materials (e.g., green waste, food waste, sewage, trash, debris, organic compounds, and natural erosion of organic materials) into surface waters. Particularly in polluted waters, bacterial degradation of organic materials can result in a sustained net decline in oxygen concentrations.

Sediment/Turbidity

Sediment is soil or dirt that can be transported and deposited as a result of wind, water, or gravity action. Rain washes soil particles off of plowed fields, construction sites, logging sites, urban surfaces, dirt roads, stream banks, and strip-mined lands into water bodies. Sediment can severely alter aquatic communities. Suspended sediment may interfere with recreational activities and aesthetic enjoyment of water bodies by reducing water clarity. Sediment in water is often measured as total suspended solids (TSS). Turbidity, the cloudiness or haziness of water, is an indicator of sediment levels.

Nutrients (Nitrogen and Phosphorus)

Potential sources of nutrients in storm water include fertilizer use, discharge of wash water that contains soaps, detergents (variety of sources including restaurants, commercial properties, and residential car washing), combined sewer overflows (CSOs), and septic systems. High nutrient concentrations may cause accelerated or excessive growth of algae and eutrophication in lakes and other water sources. Nitrogen species are often measured as total nitrogen, total kjeldahl nitrogen (TKN), ammonia, nitrate, and/or nitrate+nitrite. Phosphorus species are often measured as dissolved phosphorus and total phosphorus.

Organic Compounds (Including Pesticides and Oil and Grease)

Pesticides, polychlorinated biphenyls (PCBs), polycyclic aromatic hydrocarbons (PAHs), oil and grease, and dioxins are toxic organic compounds that are common pollutants in urban runoff and are particularly dangerous in the aquatic environment. Landscaped areas are potential sources of pesticides entering storm water. Several pesticide formulations are banned but some permissible pesticides still present toxicity risk to aquatic organisms. PCBs are a similar class of toxic organic compounds that can contaminate storm water through leaking electrical transformers. PCBs can settle in sediments of receiving waters and, like pesticide compounds, present a serious toxic threat to aquatic organisms. Dioxins are compounds that are formed through combustion, chlorine bleaching, and manufacturing processes. Some dioxin derivatives are carcinogenic to humans and toxic to aquatic life and can bioaccumulate in the food chain. Oil and grease enter storm water through a variety of mechanisms and sources, including automotive sources, leakages/spills, parking lots, restaurants, and illegal or improper disposal. Some of the hydrocarbons that are found in oil and grease are toxic to aquatic organisms and produce unsightly sheens, even at low concentrations.

Trash/Debris

The trash and debris category includes debris and floatables. Trash enters storm water through storm drain inlets, areas with high pedestrian traffic, and poor landscape maintenance practices. Not only are gross pollutants unsightly, but they may also interfere with oxygen exchange, carry bacteria, and cause vector problems.

Chloride/Road Salt

Road salt use in the United States has doubled since the 1980s, resulting in widespread salt contamination in the eastern United States. Much of the problem is caused by chloride, which is toxic to aquatic life at high concentrations and can also affect downstream vegetation. Chloride moves readily to underlying groundwater and may be one of the most important emerging contaminants for urban storm water.

Dissolved Gas Supersaturation

Supersaturation of atmospheric gases in water can be harmful to aquatic life. Specific causes include: excessive oxygenation from photosynthesis as a result of eutrophication, air entrainment in spilled (e.g., over dam spillways) or pumped water, mixing of waters of different temperatures, warm water discharges from cooling facilities, ice formation, as well as other man-made and natural causes.

Metals

In general, metals that are typically found in storm water include cadmium, chromium, copper, lead, nickel, zinc, and mercury. Potential sources include naturally occurring metals, atmospheric deposition, automobiles, illegal or improper disposal of lead batteries, mining and industrial activities, and many common materials (e.g., galvanized metal, paint, preserved wood, etc.). Metals can be toxic to aquatic organisms and can contaminate drinking water supplies.

Once the pollutants/parameters of concern have been identified based on the 303(d) list for Boone, Campbell, and Kenton Counties (Table 6.1-1), the pollutants/parameters of concern should be further refined by identifying the pollutants based on land use.

General guidance for identifying the anticipated and potential pollutants of concern based on general land use characteristics within Northern Kentucky are provided in Table 6.1-2. The actual pollutants of concern for a given site may differ from those shown, and additional pollutants of concern may be identified based on specific site characteristics, such as known soil contaminants in redevelopment sites or specific proposed site activities.

Table 6.1-2 Primary Pollutants/Parameters of Concern Based on Land Use

| LAND USE | POLLUTANT CATEGORY OF CONCERN | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | PATHOGENS | METALS | NUTRIENTS | PESTICIDES | ORGANIC COMPOUNDS (HYDROCARBONS, OIL & GREASE, SOLVENTS, PAHS) | SEDIMENTS | TRASH & DEBRIS | OXYGEN DEMANDING SUBSTANCES (GREEN AND FOOD WASTE; SEWAGE) | CHLORIDE | HYDROMODIFICATION(6) |
| Residential Development | X | P(2) | X | X | X | X | X | X | P(5) | P |
| Commercial/Institutional Development | P(1)(3) | P(2) | P(1) | P(1) | X | P(1) | X | P(1)(3)(4) | P(5) | P |
| Industrial Areas | P(1) | X | P(1) | P(1) | X | P | X | P(1)(3)(4) | P | P |
| Automotive Repair Shops | P(1) | X | P(1) | P(1) | X | P(1) | X | P(1)(4) | P(5) | P |
| Restaurants | X | P(2) | P(1) | P(1) | X | P(1) | X | X | P(5) | P |
| Parking Lots | P(1) | X | P(1) | P(1) | X | P | X | P(1)(4) | P(5) | P |
| Streets, Highways & Freeways | P(1) | X | P(1) | P(1) | X | X | X | P(1)(4) | P(5) | P |

X = anticipated
P = potential
(1) A potential pollutant if chemicals associated with landscape maintenance such as fertilizers and pesticides are employed on site.
(2) A potential pollutant if the project includes uncovered parking areas
(3) A potential pollutant if land use involves food or animal waste products
(4) A potential pollutant if combined sewer overflows, illicit sewage discharges, or septic systems exist
(5) A potential pollutant if snow removal activities are performed
(6) A potential pollutant depending on location of the project within in the region and the receiving water(s)

### *6.1.3 Step 3: Identify Appropriate BMPs to Treat Primary Pollutants of Concern*

Once the primary pollutants of concern are identified, appropriate BMPs are selected by their ability to treat these pollutants. As opposed to other design approaches that recommend the selection of typical BMPs based solely on documented performance factors, such as percent removal, effluent quality and/or percent capture, the design approach contained herein recommends the selection of BMPs through consideration of several factors. The ultimate selection of appropriate storm water treatment BMPs includes consideration of the methods of pollutant removal that address the primary pollutants of concern, consideration of other factors/constraints (described in step 4), and consideration of documented performance information (provided in Appendix A). Consideration of the method of pollutant removal is particularly applicable when designing a series of storm water treatment BMPs (i.e., forming a “treatment train”).

The methods of pollutant removal utilized for storm water treatment BMPs can be divided into four fundamental process categories: 1) hydrologic operations, 2) physical operations, 3) biological processes, and 4) chemical processes. Hydrologic operations are essentially a subset of physical operations and include the principles of flow attenuation (e.g., peak shaving and detention) and volume reduction (e.g., infiltration and evapotranspiration). Physical operations, as referred to herein, include the principles of size separation and exclusion (e.g., screening and filtration), density separation (e.g., sedimentation and flotation), aeration and volatilization, and physical agent disinfection (e.g., ultra-violet light and heat). Biological processes include the principles of microbially-mediated transformations (e.g., redox reactions resulting from microbial respiration) and uptake and storage (e.g., bioassimilation). Chemical processes include the principles of sorption (e.g., ion exchange and surface complexation), coagulation and flocculation (e.g., particle agglomeration and precipitation), and chemical agent disinfection (e.g., chlorination and ozonation). The selection of any one of these methods of pollutant removal should be based on the nature of the target pollutants and parameters (e.g. temperature and hydromodification) in relation to specific storm water management goals.

Most treatment facilities include more than one method of pollutant removal. For example, extended detention basins may reduce the total runoff volume due to infiltration and evapotranspiration (ET), as well as attenuate peak flows which help particulates to settle out. Furthermore, some BMPs can be modified to include methods of pollutant removal that are typically not incorporated in their design, such as including amended soils to promote retention and infiltration/evapotranspiration in a vegetated swale. Consequently, several BMPs may include multiple pollutant removal methods. In order to exploit the synergy amongst BMPs, the placement or order of BMPs and BMP components within a treatment system should be carefully considered. The recommended approach is to use the concept of the treatment train based on the following general progression:

(1) Minimize flow rates and/or volume of runoff (site design practices, and hydrological source controls, including within the BMP system);

(2) Remove bulk solids (> 5mm) (primary treatment);

(3) Remove settleable solids (>75 µm) and liquid floatables (primary treatment);

(4) Remove suspended (25-75 µm) and colloidal solids (> 0.1-25 µm) (secondary treatment); and,

(5) Remove colloidal, dissolved, volatile, and pathogenic constituents (tertiary treatment).

It is important to note that some storm water BMPs, such as vegetated swales, may be used as either primary and/or secondary components of a treatment train. Furthermore, tertiary treatment may be provided in BMPs that provide secondary treatment, such as constructed wetlands. Therefore, it may be more useful to categorize BMPs (and their components) according to the method of pollutant removal that they provide. Table 6.1-3 provides a guide for linking Northern Kentucky’s primary pollutants of concern and methods of pollutant removal to appropriate storm water treatment BMPs. The choice of BMP should be driven by the target pollutants and the method(s) of pollutant removal needed to address those pollutants.

Table 6.1-3 Pollutants of Concerns, Proposed BMPs, and Corresponding Methods of Pollutant Removal

| POLLUTANT OF CONCERN | PROPOSED BMP TYPES | METHOD OF POLLUTANT REMOVAL |
|---|---|---|
| ***Volume Reduction*** *(All Pollutants)* | Subsurface vaults | Infiltration |
| | Extended detention basins (extended dry pond) | Infiltration |
| | Bioretention/rain gardens | Infiltration |
| | Vegetated swales and filter strips | Infiltration |
| | Green roofs | Evapotranspiration |
| | Permeable pavements | Retention, Infiltration, and Evapotranspiration |
| | Low impact development techniques | N/A |
| ***Sediments*** *(TSS and Turbidity)* | Extended detention basins (extended dry pond) | Flocculation and Coagulation; Sedimentation |
| | Retention Basins (wet pond) | Flocculation and Coagulation; Filtration; Sedimentation |
| | Storm water wetlands | Flocculation and Coagulation; Filtration; Sedimentation; Filtration |
| | Vegetated swales and filter strips | Size Separation and Exclusion (screening); Filtration; Sedimentation (swales w/ check dams) |
| | Bioretention/rain gardens | Size Separation and Exclusion (screening); Filtration; Sedimentation |
| | Media filters | Size Separation and Exclusion (screening); Filtration; Sedimentation |
| | Hydrodynamic devices | Density, Gravity, and Inertial Separation |
| | Source control [1] and low impact development techniques | N/A |
| ***Nutrients*** *(Phosphorous and Nitrogen (Nitrate+Nitrite-N and Ammonia-N))* | Retention Basins (wet pond) | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Storm water wetlands | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Bioretention/rain gardens | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Media filters | Sorption; Ion Exchange; Filtration |
| | Source control [1] and low impact development techniques | N/A |
| ***Metals*** *(Aluminum, Copper, Lead, Mercury, and Zinc)* | Extended detention basins (extended dry pond) | Sorption; Chemical Precipitation; Sedimentation |
| | Retention Basins (wet pond) | Sorption; Chemical Precipitation; Uptake and Storage; Filtration; Sedimentation |
| | Storm water wetlands | Sorption; Chemical Precipitation; Uptake and Storage; Filtration; Sedimentation |
| | Bioretention | Sorption; Chemical Precipitation; Uptake and Storage; Filtration; Sedimentation |
| | Vegetated swales and filter strips (with amended soils) | Sorption; Chemical Precipitation; Uptake and Storage; Filtration; Sedimentation |

Table 6.1-3 Pollutants of Concerns, Proposed BMPs, and Corresponding Methods of Pollutant Removal

| POLLUTANT OF CONCERN | PROPOSED BMP TYPES | METHOD OF POLLUTANT REMOVAL |
|---|---|---|
| | Media filters | Sorption; Ion Exchange; Filtration |
| | Source control[1] and low impact development techniques | N/A |
| ***Peak Flow Control*** | Extended detention basins (extended dry pond) | Detention |
| | Retention Basins (wet pond) | Retention and Detention |
| | Storm water wetlands | Retention and Detention |
| | Underground vaults | Detention and Infiltration |
| | Permeable pavement | Retention, Infiltration, and Evapotranspiration |
| | Low impact development techniques | N/A |
| ***Pathogens*** *(Bacteria, Viruses, and Protozoa)* | Detention basin (dry pond) | Natural Disinfection (Solar Irradiation) |
| | Storm water wetlands | Natural Disinfection (Solar Irradiation) |
| | Bioretention/rain gardens | Natural Disinfection (Filtration) |
| | Media filters | Natural Disinfection (Solar Irradiation) |
| | Underground vaults w/ Infiltration | Natural Disinfection (Filtration) |
| | Ultra-violet systems | Natural Disinfection (Solar Irradiation) |
| | Chemical disinfection systems: chlorine or ozone | Chemical Disinfection |
| | Source control[1] and low impact development techniques | N/A |
| ***Organic Compounds, Pesticides, Oxygen Demanding Substances*** *(Oil and Grease, Dioxins, PCBs and, PAHs; herbicides, insecticides and, fungicides; Sewage; Food Waste; Green Waste)* | Extended detention basins (extended dry pond) | Sorption; Microbially Mediated Transformation; Sedimentation |
| | Retention basins (wet pond) | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Storm water wetlands | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Bioretention/rain gardens | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Vegetated swales and filter strips (with amended soils) | Sorption; Microbially Mediated Transformation; Uptake and Storage; Filtration; Sedimentation |
| | Media filters | Sorption; Microbially Mediated Transformation; Filtration; Sedimentation |
| | Sprinklers and aerators | Aeration and Volatilization |
| | Source control[1] and low impact development techniques | N/A |
| ***Trash & Debris*** | Screens/bars/trash racks | Size Separation and Exclusion (screening and filtration) |
| | Catch basin inserts (i.e., surficial filters) | Size Separation and Exclusion (screening and filtration) |
| | Extended detention basins (extended dry pond) | Density and Gravity Separation; Size Separation and Exclusion (outlet structure) |

Table 6.1-3 Pollutants of Concerns, Proposed BMPs, and Corresponding Methods of Pollutant Removal

| POLLUTANT OF CONCERN | PROPOSED BMP TYPES | METHOD OF POLLUTANT REMOVAL |
|---|---|---|
| | Retention Basins (wet pond) | Density and Gravity Separation; Size Separation and Exclusion (outlet structure) |
| | Storm water wetlands | Density and Gravity Separation; Size Separation and Exclusion (outlet structure) |
| | Bioretention/rain gardens | Size Separation and Exclusion; Filtration |
| | Gravity separators | Density, Gravity, and Inertial Separation; Size Separation and Exclusion |
| | Media filtration | Size Separation and Exclusion (filtration) |
| | Source control[1] and low impact development techniques | N/A |

N/A – not applicable

[1] Source control BMPs are nonstructural BMPs that prevent the release of pollutants by controlling likely sources. Examples of source controls are storm drain inlet signs that say "no dumping, drains to creek", integrated pest management practices, vehicle and equipment maintenance procedures, protection of materials stored outdoors, and spill prevention and control procedures.

### *6.1.4 Step 4: Identify Other Factors/Constraints for BMP Selection*

It is also important to note that factors other than pollutants of concern may affect BMP selection. These factors include regulatory requirements, hydromodification objectives, hydrology and hydraulics, site specific constraints (e.g., soil/bedrock types, depths to bedrock and groundwater table, contaminated soils, etc.), regional constraints, aesthetics, cost, reliability, safety, and maintenance considerations. Other factors such as natural resource planning considerations and the desire to integrate low impact development techniques may also influence which storm water treatment BMPs are most appropriate for a given project. Table 6.1-4 is a BMP practicability screening matrix that can be used to assist in the selection of BMPs for a particular site. The table briefly summarizes the critical design parameters, typical pollutants removed (including volume reduction capabilities), major constraints, and maintenance requirements. Some proprietary devices such as hydrodynamic devices are not included in Table 6.1-4 due to the wide variability in system design, operation, and maintenance requirements. However, some proprietary systems have similar design/attributes to some of the BMP types below when selected and sized properly.

Table 6.1-4 BMP Practicability Screening Matrix

| BMP TYPE | CRITICAL DESIGN PARAMETERS | TYPICAL POLLUTANTS REMOVED | MAJOR CONSTRAINTS | MAINTENANCE REQUIREMENTS |
|---|---|---|---|---|
| Extended Detention Basin (Dry Pond) | Stage-discharge relationship and drain time (outlet design); Storage capacity; Length to width ratio; Location of inlets and outlets; Flow rate diversion for off-line facilities | High removal efficiency of coarse solids, trash and debris; Moderate removal of suspended sediment; Little to no predicted removal of dissolved metals and nutrients; Moderate volume reduction | Surface space availability; Depth of excavation; Slope stability; Compatibility with flood control | Dredging of forebay required approximately every 5 years with reestablishment of pond bottom; Frequent mowing; Side slope upkeep; Trash and debris removal; Periodic inspections |
| Retention Basin (Wet Pond) | Length to width ratio; Stage- discharge relationship; Permanent pool and surcharge capacity; Maximum depth; Base flow; Plant selection; Flow rate diversion for off- line facilities | High removal efficiency of coarse solids, suspended solids, trash, and debris; Some removal of dissolved solids, total phosphorus, soluble nutrients, trace metals, coliform and organics; Low volume reduction | Surface space availability; Depth of excavation; Slope stability; Compatibility with flood control; Vector control | Dredging required approximately every 5 years with reestablishment of pond bottom; Side slope upkeep; Trash and debris removal; Periodic inspections; Removal of algal mats and control of fringe vegetation |
| Storm Water Wetland | Volume of design storm; Length to width ratio; Depth distribution; Base flow; Plant selection; Flow rate diversion for off- line facilities | High removal efficiency of coarse solids, suspended sediment, trash and debris; Moderate removal of metals; Good to moderate removal of phosphorus/nitrogen; Variable removal of indicator bacteria; Low volume reduction | Surface space availability; soil type; System hydraulics; Vector control; Lack of base flow | Monthly inspections required until vegetation is established; Periodic removal of nuisance species and litter as required; vector control |
| Vegetated Swale | Retention time; Minimum length; Maximum width; Flow rate, velocity, and depth; No. of check dams; Grass selection | High removal efficiency of coarse solids, trash, and debris; Moderate removal of suspended sediment; Variable removal of nutrients and metals; Moderate volume reduction | Steep terrain; Availability of pervious area; Size of tributary area; High flows | Seasonal mowing and vegetation upkeep required; Sediment removal when exceeds 4 inches in any location; Periodic inspections |
| Filter Strip | Retention time; Minimum length; Longitudinal slope; Flow rate, | High removal efficiency of coarse solids, trash, and debris; Moderate removal | Steep terrain; Availability of pervious area; Ability | Seasonal mowing and vegetation upkeep required; Sediment |

Table 6.1-4 BMP Practicability Screening Matrix

| BMP TYPE | CRITICAL DESIGN PARAMETERS | TYPICAL POLLUTANTS REMOVED | MAJOR CONSTRAINTS | MAINTENANCE REQUIREMENTS |
|---|---|---|---|---|
| | velocity, and depth; Grass selection | of suspended sediment; Limited removal of nutrients and dissolved metals; Moderate volume reduction | to maintain sheet flow; Size of tributary area; High flows | removal when exceeds 4 inches in any location; Periodic regrading and reseeding; Periodic inspections |
| Media Filter | Maximum emptying time; Media type and volume; Particle size gradation; Depth to groundwater | High removal efficiency of coarse solids, suspended sediment, and metals; Some removal of nutrients and BOD; Low volume reduction (not representative of bioretention areas, see below) | Vertical relief and proximity to storm drain; Large drainage area; High sediment loadings; Aesthetics | Seasonal surface scarification; Periodic removal of trash and debris and accumulated silt on bed surface (when >0.5" thick); Frequent inspection; Potential media or cartridge replacement |
| Bioretention/ Rain Gardens | Soil characteristics and amendments; Depth to groundwater; Area and ponding depth; Storage capacity; Plant selection | High removal efficiency of coarse solids, trash, and debris; Moderate removal of suspended sediment, metals, and bacteria; Variable removal of nutrients; Low to high volume reduction | Field infiltration rate; Depth to groundwater; Contaminated soils; Proximity to storm drain; Vertical relief and proximity to storm drain; Surface space availability | Semi-annual/annual, and post-storm inspections; Vegetation upkeep; Periodic surface scarification; Sediment removal |
| Permeable Pavement | Pavement selection; Soil characteristics; Infiltration rate; Drawdown time for gravel drainage layer; Depth to groundwater | High removal efficiency of sediment; Moderate removal of metals, oils and grease, nutrients, bacteria, and peak flow control; Low removal of trash and debris; Low to medium volume reduction | Field infiltration rate; Drawdown time for gravel storage layer; Proximity to storm drain; Size of tributary area | Semi-annual/annual, and post-storm inspections; Remove sediment accumulation (vacuuming); Stabilize adjacent vegetative areas |
| Subsurface Vaults | Min/Max infiltration rate; Depth to groundwater; Storage capacity | High removal efficiency of coarse solids, particulate and suspended sediment; Moderate removal of phosphorus/nitrogen; Dissolved metals and pathogen removal dependent on soil types; High volume reduction | Underground utility conflicts; Field infiltration rate; Depth to groundwater; Contaminated soils; Proximity to structures | Semi-annual/annual and post-storm inspections; and sediment removal |
| Green Roofs | Media characteristics; | Low removal efficiency of | Cost; Structural | Semi-annual/annual, |

Table 6.1-4 BMP Practicability Screening Matrix

| BMP TYPE | CRITICAL DESIGN PARAMETERS | TYPICAL POLLUTANTS REMOVED | MAJOR CONSTRAINTS | MAINTENANCE REQUIREMENTS |
|---|---|---|---|---|
| | Media depth; Storage capacity; Plant selection | metals and nutrients; Moderate volume reduction | strength of building | and post-storm inspections; Vegetation upkeep; and debris removal; Potential replacement of media, drainage layer, water proofing membrane, or vegetation |
| Gravity Separators | Treatment rate; length to width ratio | High removal efficiency of oil and grease and coarse solids; Moderate removal of suspended solids, phosphorus and organic nitrogen. | Underground utility conflicts; Vertical relief and proximity to storm drain | Semi-annual/annual, and post-storm inspections; Trash and debris removal |

### *6.1.5 Step 5: Select Treatment BMPs Based on Primary Pollutants of Concern, Method of Pollutant Removal for Targeting Primary Pollutants of Concern, Other Factors/Constraints, and BMP Performance*

Many factors affect BMP selection. All of the information from Steps 1-5 should be used for identifying the most appropriate treatment BMPs for a specific project with the primary objective being to target the primary pollutants of concern. If treatment BMP performance information is unavailable or not applicable, the designer should target the primary pollutants of concern based on the methods of pollutant removal that provide effective removal of the target pollutants.

While specific pollutant concentration and reduction requirements are not currently enforced in the Northern Kentucky community, future regulations are expected to set pollutant removal standards for new and redevelopment projects. For this purpose, detailed information relating to BMP performance and pollutant removal research is included in Appendix A.

### *6.1.6 Example BMP Selection Using the 5-Step Method*

A 10-acre residential development along Phillips Creek has been proposed. Use the 5-step method to identify BMPs that will minimize hydraulic impacts on Phillips Creek while incorporating treatment for pollutants of concern.

- Step 1: The project objectives are as follows:
    - Volume reduction and Peak Shaving ("minimize hydraulic impact"); and
    - Treat pollutants of concern.
- Step 2: The primary pollutant of concern (from Table 6.1-1) is fecal coliform. From Table 6.1-2, pathogens (which include the indicator organism fecal coliform) are anticipated pollutants from residential developments. Based on Step 1, any BMP should be capable of treating fecal coliform.
- Step 3: Table 6.1-3 lists six BMP types that can be used to treat pathogens including extended

detention basins (shallow ponds are more effective) and bioretention/rain gardens, storm water wetlands, UV systems, and chemical disinfection systems. Of these BMPs, only extended detention basins, in areas of permeable soils, also achieve the stated hydraulic goals. However, a treatment train approach is more likely to be successful. For instance, use bioretention for volume reduction leading to an extended detention basin via a vegetated swale. The bioretention and the vegetated swale meet the goals of volume reduction. The extended detention basin meets the goal of peak shaving and fecal coliform reduction. A number of treatment trains can be envisioned with each component of the treatment train meeting at least one of the objectives identified in Step 1.

- Step 4: Use Table 6.1-4 to assess the practicability of each of the treatment trains. Treatment trains that work in one area may be inappropriate in another due to differences in soil type, surface slope, space availability, or a number of other reasons.
- Step 5: The BMPs, or more likely, treatment trains created in Step 3 that survived the exclusion process of Step 4 are good candidates for implementation. The final selection will likely be determined by capital costs and maintenance issues.

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CITY OF FLORENCE KENTUCKY

## 7.1 BMP FACT SHEET OVERVIEW

The purpose of the BMP Fact Sheets is to provide guidance on the applicability, design requirements, and sizing standards for volume control and quality control BMPs. The following BMPs are included in this chapter:

- Biofiltration Swale;
- Bioretention/Rain Garden;
- Extended Detention Basin (Dry Pond);
- Gravity Separator;
- Green Roof;
- Media Bed Filter;
- Permeable Pavement;
- Planter Box
- Retention Basin (Wet Pond);
- Storm Water Wetland;
- Street Trees;
- Subsurface Vault; and
- Vegetated Filter Strip.

Each BMP Fact Sheet provides the following information:

- BMP description and applicability;
- Performance data;
- Advantages/limitations;
- Site suitability considerations;
- Design criteria;
- Sizing and design procedures;
- Example design schematics; and
- Maintenance information.

## 7.2 WATER QUALITY BMP DESIGN CONSIDERATIONS

The selection of individual water quality BMPs will vary depending upon the results of the BMP selection procedure described in Chapter 6. Following the BMP selection process, several design components for the majority of water quality BMPs should be considered during the initial phases of design. These key design considerations are not all-inclusive but are described as a preface to the BMP Fact Sheets because they tend to be routinely included in most water quality BMP designs. This introduction to Chapter 7 will focus on the following key design considerations:

- Erosion Control;
- Choker Layer;
- Energy Dissipation;
- Watertight Control Measures; and
- Waterfowl and Mosquito Control.

### *7.2.1 Erosion Control*

The functionality of any BMP is highly dependent upon proper site stabilization and adequate erosion prevention and sediment control. Erosion prevention and sediment control measures should be adequate enough to prevent sedimentation from flowing onto a water quality BMP, and should be checked on a routine basis during construction. Careful consideration should be given to construction sequencing on sites with water quality BMPs to help prevent situations during construction where construction activities tributary to a BMP could compromise the functionality of the control. For example, if a rain garden that has a planting media with a designed infiltration rate of 8 inches per hour is constructed before the remainder of the site is stabilized, storm water runoff could deposit sediment on the top surface of the planting media and diminish the ability of the rain garden to infiltrate. Construction oversight with an emphasis on proper maintenance of erosion prevention and sediment control measures will help protect the integrity of the water quality BMP throughout the construction phase.

Figure 7.2-1 Erosion prevention and sediment control measures (sand bags) used to prevent sediment from flowing into rain garden.

### 7.2.2 Choker Layer

Several of the water quality BMPs described in Section 7 make reference to a choker layer or filter layer between the planting media and gravel storage layer. The intent of a choker layer is to prevent soil particles in the planting media from migrating into the underlying gravel storage layer and underdrain system. Any soil particles that migrate into the gravel layer could diminish the storage capacity achieved in the void spaces of the gravel, and could also result in clogging issues of the underdrain system. Several options for the choker layer may be considered, including the following:

- Non-woven geotextile filter fabric (refer to specific BMP Fact Sheets for material requirements).
- 2 to 4 inches of washed sand underlain with 2 inches of choking stone (typically No. 8 or No. 89 pea gravel).
- Thin layer of choking stone over top of a non-woven geotextile filter fabric.

Each of the options above has advantages and disadvantages, but insufficient studies for each have yet to be published to document long-term performance, such as potential for clogging over time. Therefore, the choker layer option should be selected by the design engineer depending upon the specific design configuration of the water quality BMP. Additional information regarding the choker layer is included in several of the BMP Fact Sheets.

Figure 7.2-2 Installation of non-woven geotextile filter fabric between planting media and drain rock.

### 7.2.3 Energy Dissipation

In some circumstances, it may be necessary to convey storm water flows to a water quality BMP through a point source discharge, such as a storm sewer that discharges directly into a BMP. For these situations, adequate energy dissipation will be needed to prevent erosion of the water quality BMP. Typical energy dissipation techniques may include rip-rap, turf reinforcement mats, drop manholes, etc. The BMP Manual does not include information on the types of energy dissipation or design criteria, but this is a key design consideration that must be evaluated in most circumstances to provide adequate stability of the water quality BMP at the point source discharge, which could maintain the long-term performance of the BMP.

Figure 7.2-3 Energy dissipation (rip-rap) at point of discharge into bioretention basin.

### *7.2.4 Watertight Control Measures*

Because most water quality BMPs are intended to store, infiltrate, and slowly release storm water runoff, designs should incorporate watertight control measures to minimize potential for short-circuiting the BMP. This design consideration is particularly important in scenarios where a gravel storage layer is hydraulically connected to the granular backfill material of an underdrain or storm sewer that exits the BMP. Anti-seep collars, or bedding dikes with low permeability (e.g., bentonite backfill, concrete, compacted clayey backfill, etc.) should be provided where the underdrain pipe or storm sewer exits the water quality BMP. The anti-seep collar will help minimize any storm water flow through the granular backfill and bedding and will improve the ability of the water quality BMP to retain storm water without unintended leaking.

### *7.2.5 Waterfowl and Mosquito Control*

The presence of waterfowl can become a problem in storm water basins, ponds, and wetlands; and management should be considered as early as the design phase to be most effective. These species create potential health concerns for humans as waterfowl bacteria can contaminate waterbodies and are known human pathogens. Thick vegetation around the perimeter of these basins has proven to be the single-most effective deterrent of Canada geese, while overwater grid wires and other barriers have also proven to be effective for geese and mallards. By implementing multiple controls, waterfowl will be even more deterred from these locations. Additional information on the presence and management of waterfowl can be found in the Technical Memorandum, written by LimnoTech, included in Appendix J.

Mosquitos are another nuisance that can accompany storm water basins, ponds, and wetlands. Mosquitos tend to favor shallow, stagnant waters for development; conditions which can be frequently found in these basins. Mosquitos are also tolerant of poor water quality. Design of basins and wetlands should reduce standing water that is less than 12” deep and should maximize circulation throughout the basin to avoid stagnant areas. Additional information on the presence and management of mosquitos can be found in the Technical Memorandum, written by LimnoTech, included in Appendix J.

## 7.3 OTHER CONSIDERATIONS

In addition to requiring the construction of water quality BMPs for new and redevelopment the Phase II Storm Water Regulations also require SD1 and the City of Florence to conduct post-construction inspections, enforce maintenance of BMPs, and demonstrate and document that post-construction BMPs have been installed per the design specifications. The following Appendices provide additional information on these topics:

- Appendix G: BMP Inspection and Maintenance Checklists;
- Appendix H: Post-Construction Storm Water Controls Maintenance Agreement; and
- Appendix I: Post-Construction Storm Water Controls Installation Certification.

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# BIOFILTRATION SWALE

Structural Best Management Practice

BIOFILTRATION SWALE

Volume Control ☑

Quality Control ☑

Virginia Department of Conservation and Recreation

| PERFORMANCE | | | | |
|---|---|---|---|---|
| M | Sediment | | L | Bacteria |
| M | Metals | | M | Trash and debris |
| M | Oil and grease | | L | Volume Reduction |
| L | Nutrients | | M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

*Applications*

- Commercial and institutional
- Residential/subdivision
- Multi-family and mixed use
- Parking lots
- Road shoulders and medians
- Parks and golf courses
- Pretreatment for other BMPs

*Advantages*

- ✓ Combines storm water treatment with runoff conveyance
- ✓ Often less capital cost than hardened conveyance structures
- ✓ Suspended solids and particulate-bound pollutant removal
- ✓ Volume & peak flow reduction
- ✓ Low cost per drainage area
- ✓ Aesthetically pleasing

*Limitations*

- Higher maintenance than curb and gutter
- Limited removal of dissolved pollutants and nutrients
- Less suitable for large drainage areas
- Risk of sediment re-suspension when conveying flood control design flow rates

## DESCRIPTION

Biofiltration swales are vegetated storm water conveyances that treat runoff by filtration, shallow sedimentation, and infiltration. Additional minor removal mechanisms include biochemical processes in the underlying planting media such as adsorption and microbial transformations of dissolved pollutants. If designed as on-line drainage system features capable of conveying peak flow rates, biofiltration swales can provide downstream channel and flood protection. However, on-line biofiltration swales are more vulnerable to re-suspension of captured sediment if not carefully designed and maintained. When properly incorporated into an overall site design, swales may reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

An effective biofiltration swale aims to provide uniform sheet flow through a densely vegetated area (bottom of swale) for a period of 5-9 minutes. The type of vegetation in the swale can vary depending on its location within a development project and is a function of designer choice and project objectives.

## SITE SUITABILITY CONSIDERATIONS

Swales have a wide range of applications and can be used in highway, residential, commercial, institutional, and industrial areas for conveyance and treatment of runoff from roads, parking lots, rooftops, and other impervious surfaces. Swales are more effective on sites that allow continuous flow with minimal interruption from driveway culverts or other obstacles. It is recommended that driveways be at least 30 feet apart if swales are to be used in residential applications. Also, swales treating larger areas may require excessively wide bottom widths to provide adequate treatment at the water quality design flow rate. Swales should have a flat cross-sectional bottom and widths should be generally less than 7 feet to promote uniform flow depths. The following table summarizes general site suitability considerations for biofiltration swales.

| SITE SUITABILITY CONSIDERATIONS FOR BIOFILTRATION SWALES | |
|---|---|
| Tributary Area | < 5 acres (217,800 $ft^2$) |
| Typical BMP area as percentage of tributary area (%) | < 5 percent |
| Site slope (%) | 1 to 6 percent [2] |
| Minimum distance between culverts | 30 ft |
| Depth to seasonally high groundwater table below swale bottom | < 5 ft use underdrains<br>> 5 ft underdrain not required |
| Hydrologic soil group | Any [2] |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.

2 – If the swale is located 10 feet from a building or foundation, has a longitudinal slope less than 1.5%, or has poorly drained soils (hydrologic soil groups “C” or “D”), underdrains should be incorporated. If underdrains are provided, site must have adequate relief between land surface and the storm water conveyance system to permit vertical percolation through the gravel drainage layer (open-graded base/sub-base) and underdrain to the storm water conveyance system.

The effectiveness of a biofiltration swale is directly related to the contributing land use, the size of the drainage area, the soil type, slope, drainage area imperviousness, proposed grasses, and the swale dimensions. Natural low points in the topography are well-suited as swale locations, as are natural drainage courses, although infiltration capability may be reduced in these situations. The topography of a site should allow for the design of a swale with sufficiently mild slope and flow capacity. Swales are impractical in areas of extreme (very flat or steep) slopes. Swales are ideal as an alternative to curbs and gutters within parking lots and along roadside rights-of-way in gently sloping terrain. Additional site suitability recommendations and potential limitations for biofiltration swales are listed below.

- Placement – Placement of biofiltration swales should take into account the location and function of other site features (buffers, undisturbed natural areas, etc.). Placement should also attempt to aesthetically fit the swale into the landscape as much as possible. Sharp bends in swales should be avoided or bank armoring should be provided to protect from scour.

- Soils – Where possible, construct swales in areas of uncompacted cut. Avoid constructing side slopes in fill material, which can be prone to erosion and/or structural damage by burrowing animals, if possible. Swales should either be lined or avoided in areas where soils might be contaminated or highly erodible.

- Development density – Implementing biofiltration swales is challenging when development density exceeds four dwelling units per acre, in which case the number of driveway culverts often increases to the point where swales essentially become broken-pipe systems.

- Length – Swales typically require at least 100 feet in length if used to meet the water quality treatment requirements. The swale can be shorter than 100 feet if it is used for pretreatment or for flow conveyance only.
- Adjacent Land Uses – Swales may not be suitable for locations that are adjacent to industrial sites or locations where the potential for spill or release of hazardous substances may occur. Sensitivity to surrounding land uses is dependent upon the design of the swale such as the filtration and infiltration capabilities of the swale.
- Shade – Areas with excessive shade may result in poor vegetative growth. For moderately shaded areas, shade tolerant plants and grasses shall be used. Excessive tree debris may smother grass or impede flow through the swale.

## DESIGN CRITERIA

Biofiltration swales can be designed to be either on-line or off-line. On-line swales are used for conveying high flows as well as providing treatment of the water quality design flow rate, and can replace curbs, gutters, and other storm drain infrastructure. Off-line swales are the preferred practice from a water quality treatment perspective; however, off-line swales may not always be feasible or desirable given system objectives and site constraints. If a swale is on-line, then the design should ensure peak flow velocities are minimized to avoid scouring and re-suspension of captured sediment. The following table summarizes the minimum design criteria for biofiltration swales. Additional sizing criteria and design guidance is provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Flood control design flow rate, $Q_{fc}$ | cfs | See SD1's Storm Water Rules and Regulations or Boone County's Subdivision Regulations for calculating $Q_{fc}$ |
| Water quality design flow rate, $Q_{wq}$ | cfs | Required for on-line and off-line swales. See Chapter 3 for calculating $Q_{wq}$ |
| Minimum bottom width | ft | 3 |
| Maximum bottom width | ft | 7; if greater than 7 use swale dividers |
| Maximum channel side slope | H:V | 3H:1V for vegetated side slopes |
| Minimum slope in flow direction | % | 1 (provide underdrains for slopes between 1% and 1.5% that have poorly drained soils – hydrologic soil group "C" or "D". ) |
| Maximum slope in flow direction | % | 6 |
| Maximum flow velocity | ft/s | 1 (water quality treatment); 3 (flood conveyance) |
| Maximum depth of flow for water quality treatment | in | 4-6 (ideal flow depth is 2 inches less than vegetation height) |
| Minimum residence (contact) time | min | 5 minimum, 9 preferred (average for water entering swale) |
| Vegetation type | -- | Varies (see Vegetation section below and Appendix C) |
| Vegetation height | in | 4 to 8 (trim or mow to maintain height) |

### *Cross-Sectional Geometry and Size*

- In general, trapezoidal channel shape shall be assumed for sizing calculations, but a more naturalistic (e.g., parabolic) channel cross-section is preferred. Trapezoidal channels become parabolic over time with sediment accumulation.
- If swale is an on-line storm water conveyance feature, it shall be sized to provide conveyance for the flood control design flow rate, $Q_{fc}$, with at least six inches of freeboard per SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.
- If swale is an offline water quality treatment swale it shall be designed to convey the flow-based water quality design flow rate, $Q_{wq}$, by using a flow diversion structure(s) (e.g., flow splitter, curb-cuts, etc.) which diverts the $Q_{wq}$ to the off-line vegetated swale designed to handle $Q_{wq}$. Freeboard for off-line swales is not required, but shall be provided if space is available.
- Mild side slopes are necessary for mowed turf swales and on-line swales used for flood control. The maximum allowable side slope for vegetated swales is 3H:1V with a preferred side slope of 4H:1V.
- Overall depth from the top of the side slope to the swale bottom shall be at least 12 inches.
- Swale length shall be sized to achieve a minimum of 5 minute hydraulic residence time (9 is preferred) for $Q_{wq}$.

- The minimum swale bottom width shall be 3 feet to allow for ease of mowing. The maximum swale bottom width shall be limited to 7 feet, unless a dividing berm is provided, then maximum bottom width can be 14 feet. Swale width is calculated without the dividing berm.
- Gradual meandering bends in the swale are desirable for aesthetic purposes and to promote slower flow.

### *Bottom Slope*

- The longitudinal slope (along the direction of flow) shall be between 1% and 6%. Longitudinal slopes between 1% and 4% are generally recommended for swales. With Northern Kentucky's topography sometimes necessitating steeper slopes, turf reinforcement mats (TRMs) can be used to reduce the energy gradient and erosion potential. Slopes should not be more than 6%, and peak velocities should not reach more than 4 feet per second for up to the 10-year storm event.
- The lateral (horizontal) slope at the bottom of the swale shall be zero (flat) to discourage channeling.

### *Water Depth and Low Flow Drain*

- For water quality design flow rate, $Q_{wq}$, water depth shall not exceed 6 inches or 2 inches below the average height of the maintained vegetation, whichever is less.
- For flood control design flow rate, $Q_{fc}$, swales should hold a maximum flow depth of 18 inches at the end point of the channel, with a 12-inch average ponding depth maintained throughout.
- If persistent dry weather base flows to the swale are expected, install a low flow drain extending the entire length of the swale. The drain shall have a minimum depth of 6 inches, and a width no more than 5% of the calculated bottom swale width; the width of the drain shall be in addition to the required bottom width.

### *Inflows and Energy Dissipation*

- Runoff can be directed into biofiltration swales either as concentrated flows or as lateral sheet flow drainage. Both are acceptable provided sufficient stabilization or energy dissipation and flow spreading is provided. If flow is to be directed into a swale via curb cuts, provide a 2 to 3 inch drop at the interface of pavement and swale. Curb cuts should be at least 12 inches wide to prevent clogging and should be spaced appropriately. The slope of the back curb should be 2 to 3% to guard against sediment aggradation and eventual blockage of flow.
- A flow spreader shall be used at the inlet so that the entrance velocity is quickly dissipated and the flow is uniformly distributed across the whole swale. Energy dissipation controls shall be constructed of sound materials such as stones, concrete, or proprietary devices that are rated to withstand the energy of the influent flows.
- If check dams are used to reduce the longitudinal slope, a flow spreader shall be provided at the toe of each vertical drop, with specifications described below.
- The maximum flow velocity under the water quality design flow rate shall not exceed 1.0 foot per second.
- The maximum flow velocity during the flood control design storm event shall not exceed 4.0 foot per second. This can be accomplished by:
    - Splitting roadside swales near high points in the road so that flows drain in opposite directions, mimicking flow patterns on the road surface.

- Limiting tributary areas to long swales by diverting flows throughout the length of the swale at regular intervals, to the downstream storm water conveyance system.

### *Underdrains*

- If underdrains are required, then they must be made of perforated or slotted, polyvinyl chloride (PVC) pipe conforming to ASTM D 3034 or equivalent or corrugated high density polyethylene (HDPE) pipe conforming to AASHTO 252M or equivalent. Underdrains shall slope at a minimum of 0.5 percent, and smooth and rigid PVC pipes shall be used as underdrains with slopes of less than 2 percent.
- The perforations or slots shall be sized to prevent the migration of the drain rock into the pipes, and shall be spaced such that the pipe has a minimum of 1 square inch of opening per lineal foot of pipe.
- The underdrain pipe must have a 6-inch minimum diameter, so it can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe must be placed at the terminal ends of the underdrain. The cleanout risers shall be plugged with a lockable well cap. It is recommended to keep the cap locked in areas prone to vandalism.
- The underdrain shall be placed parallel to the swale bottom. The underdrain shall be bedded with 6 inches of drain rock and backfilled with a minimum of 6 inches of drain rock around the top and sides of the underdrain. The drain rock shall consist of clean, washed No. 57 stone, conforming to the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet, or an approved equal, that meets the gradation requirements listed in the table below.

| SIEVE SIZE | PERCENT PASSING |
|---|---|
| 1-$^1/_2$ inch | 100 |
| 1 inch | 95-100 |
| $^1/_2$ inch | 25-60 |
| US No. 4 | 0-10 |
| US No. 8 | 0-5 |

- The drain rock must be separated from the native soil layer below and to the sides with an approved non-woven geotextile fabric. The drain rock shall be separated from the planting media above with an approved non-woven geotextile fabric or with an appropriately graded granular filter. The graded granular filter should consist of 2 to 4 inches of washed sand underlain with a minimum 2 inches of choking stone (washed No. 8 or No. 89 pea gravel). The non-woven geotextile filter fabric should not impede the infiltration rate of the planting media and should have a minimum flow rate of 50 gal/min/ft$^2$. Unless otherwise approved, the non-woven geotextile fabric shall conform to the Type II Fabric Geotextiles for Underdrains described in the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet. The minimum requirements for the non-woven geotextile filter fabric are listed in the table below.

| GEOTEXTILE PROPERTY | VALUE | TEST METHOD |
|---|---|---|
| Grab Strength (lbs.) | 80 | ASTM D4632 |
| Sewn Seam Strength (lbs.) | 70 | ASTM D4632 |
| Puncture Strength (lbs.) | 25 | ASTM D4833 |
| Trapezoid Tear (lbs.) | 25 | ASTM D4533 |
| Apparent Opening Size US Std. Sieve | No. 50 | ASTM D4751 |
| Permeability (cm/s) | 0.010 | ASTM D4491 |
| UV Degration at 150 hrs. | 70% | ASTM D4355 |
| Flow Rate (gpm/ft$^2$) | 50 | ASTM D4491 |

- The underdrain pipe must drain freely to an acceptable discharge point.
- If no underdrains are present, an observation well extending at least 5 feet into native soil below the facility is recommended to assist with identifying drainage problems.

### *Swale Divider*

- If a swale divider is used, the divider shall be constructed of a firm material that will resist weathering and not erode, such as concrete, plastic, or compacted soil seeded with grass. Treated timber or galvanized metal shall not be used. Selection of divider material must take into account maintenance activities, such as mowing.
- The divider must have a minimum height of 1 inch greater than the water quality design water depth.
- Earthen berms shall be no steeper than 3H:1V.
- Material other than earth shall be embedded to a depth sufficient to be stable.

### *Soils*

- The soil base for a biofiltration swale must provide stability and adequate support for proposed vegetation.
- When using existing site soil, it is recommended to rototill and amend the soil prior to seeding. Unless the organic content is already greater than 10%, swale soils shall be amended with 2 inches of weed free and well-aged compost. The compost shall be mixed into the native soils to a depth of 6 inches to prevent soil layering and washout of compost. The compost will contain no sawdust, green or under-composted material, or any toxic or harmful substance. It shall contain no un-sterilized manure, which can lead to high levels of pathogen indictors (coliform bacteria) in the runoff. The compost shall be free of stones, stumps, roots or other similar objects larger than 3/4 inches.
- When the existing site soil is deemed unsuitable (clayey, rocky, coarse sands, etc.) to support dense grass, replacing (rather than just amending) the top 6 inches with a bioretention soil mix is recommended. See Appendix B for example bioretention soil mixes.
- If a biofiltration swale is used for volume control, amended soils are necessary as part of the design. See Appendix B for example bioretention soil mixes.

### *Vegetation*

- Swales must be vegetated in order to provide adequate treatment of runoff via filtration. Vegetation, when chosen and maintained appropriately, also improves the aesthetics of a site. It is important to maximize water contact with vegetation and the soil surface.
- By incorporating into site landscaping, swales can be integrated into the overall site design without unnecessary loss of usable space. Tree plantings should allow enough light to pass to sustain a dense ground cover. Trees or shrubs may be used in the landscape as long as they do not over-shade the turf.
- The swale area shall be appropriately vegetated with a mix of erosion-resistant plant species that effectively bind the soil. At a minimum, the swale shall be appropriately vegetated with dense grass. It is recommended that other low growing plants that thrive under the specific site, climatic, and watering conditions shall be specified in addition to dense grasses. A mixture of dry-area and wet-area grass species that can continue to grow through silt deposits is most effective. Native or adapted grasses are preferred because they generally require less fertilizer, limited maintenance, and are more drought resistant than exotic plants. Reference Appendix C for recommended plant lists.
- If the swale is treating runoff from areas where deicing salts are applied, salt tolerant vegetation may be needed.
- When appropriate, swales that are integrated within a project may use turf or other more intensive landscaping, while swales that are located on the project perimeter, within a park, or close to an open space area are encouraged to be planted with a more naturalistic (i.e., native) plant palette.
- Irrigation is required if the seed is planted in spring or summer. Drought-tolerant grasses shall be specified to minimize irrigation requirements.
- Vegetative cover shall be at least 4 inches in height and grasses shall be no taller than 8 inches. Swale water depth will ideally be maintained 2 inches below the height of the grass and shall not exceed 6 inches.
- Prohibited non-native plant species will not be permitted. For information on invasive plant species in Kentucky, go to the Early Detection & Distribution Mapping System at http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky

## DESIGN PROCEDURE

The flow capacity of a biofiltration swale is a function of the longitudinal slope (parallel to flow), the resistance to flow (e.g., Manning's roughness), and the cross-sectional area. The cross-section is approximately trapezoidal and the area is a function of the bottom width and side slopes.

### *Step 1: Design Flow Rates*

The water quality design flow rate, $Q_{wq}$, shall be determined using the procedure provided in Chapter 3. If the swale is on-line, the flood control design flow rate, $Q_{fc}$, must also be determined using the procedure provided in SD1's Storm Water Rules and Regulations and Boone County Design Standards for Subdivision Review.

### *Step 2: Depth and Retention Time Requirements*

Select a water quality design depth and retention time based on the permissible ranges for swales shown in the Design Criteria table above. It is recommended to start with a 2-inch (0.167 ft) water quality depth, $D_{wq}$, and a 9 minute water quality retention time, $t$. To achieve permissible values for the dimensions below, these initial values may need to be altered.

### *Step 3: Bottom Width*

Compute the bottom width of the swale using the following simplified form of the Manning's equation (side slopes neglected):

$$W = \frac{n \cdot Q_{wq}}{1.49 \cdot D_{wq}^{1.67} \cdot S^{0.5}}$$

Where:

$W$ = channel bottom width (ft)
$n$ = Manning's roughness coefficient for shallow flow conditions (unit less); use 0.25.
$Q_{wq}$ = water quality design flow (cfs)
$D_{wq}$ = water quality flow depth (ft)
$S$ = longitudinal slope (ft/ft)

If the bottom width is calculated to be between 3 and 7 feet, proceed to Step 4. If bottom width is less than 3 feet, set $W$ = 3 feet and recalculate the water quality design flow depth ($D_{wq}$):

$$D_{wq} = \left[ \frac{n \cdot Q_{wq}}{1.49 \cdot W \cdot S^{0.5}} \right]^{0.6}$$

Where:

$W$ = channel bottom width (ft); use 3 feet
$n$ = Manning's roughness coefficient for shallow flow conditions (unit less); use 0.25.
$Q_{wq}$ = water quality design flow (cfs)
$D_{wq}$ = water quality flow depth (ft)
$S$ = longitudinal slope (ft/ft)

If bottom width is more than 7 feet, increase longitudinal slope (s), increase design flow depth ($D_{wq}$) to a maximum of 0.33 ft (4 in), install flow divider and flow spreader, or relocate swale downstream of a detention facility.

### Step 4: Flow Velocity

Compute the water quality design velocity, $V_{wq}$, using the bottom width and neglecting side slopes:

$$V_{wq} = \frac{Q_{wq}}{W \cdot D_{wq}}$$

Where:

$V_{wq}$ = water quality design velocity (ft/s)
$Q_{wq}$ = water quality design flow (cfs)
$W$ = channel bottom width (ft); use 3 feet
$D_{wq}$ = water quality flow depth (ft)

If $V_{wq}$ is greater than 1 ft/s, go back to Step 3 and modify longitudinal slope, bottom width (need flow divider if >8 feet), or depth. $V_{wq}$ is less than 1 ft/s proceed to Step 5.

### Step 5: Swale Length

Compute the minimum length of the swale:

$$L = t \cdot V_{wq}$$

Where:

L = minimum length of the swale (ft)
$V_{wq}$ = water quality design velocity (ft/s)
$t$ = residence time (seconds); 5 minutes (300 sec) minimum; >9 minutes (540 sec) preferred.

### Step 6: Check Flood Control Conveyance Requirements (if on-line)

Compute the flood capacity of the swale at peak allowable flow velocity and max design depth (excluding freeboard requirements):

$$Q^*_{fc} = V^*_{fc}\left(W \cdot D^*_{fc} + Z \cdot {D^*_{fc}}^2\right)$$

Where:

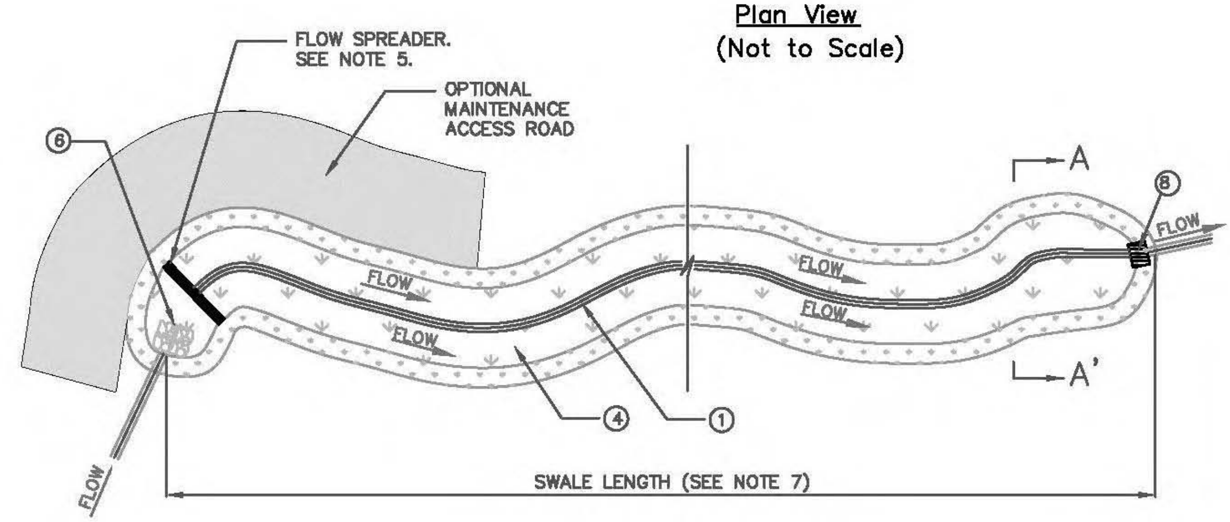
$Q^*_{fc}$ = flood capacity of the swale (cfs)
$V^*_{fc}$ = max allowable velocity in the swale (ft/s) [use 4 ft/s]
$W$ = channel bottom width (ft)
$D^*_{fc}$ = max design flow depth (ft) [≤ 2 ft excluding freeboard requirements]
$Z$ = horizontal component of the side slope (unit less)

If $Q^*_{fc} > Q_{fc}$, then increase $D^*_{fc}$ (if not already at max) or go back to Step 3 and modify longitudinal slope or bottom width (need flow divider if >7 feet). If $Q^*_{fc} \leq Q_{fc}$, then swale sizing is complete.

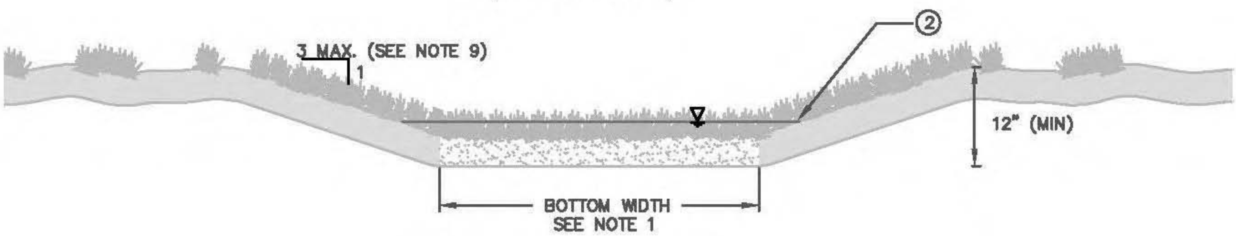
## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of biofiltration swales. Other designs are permissible if minimum design criteria are met.

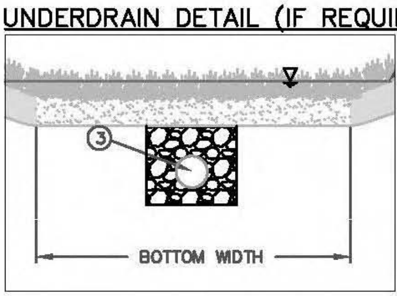
Plan View
(Not to Scale)



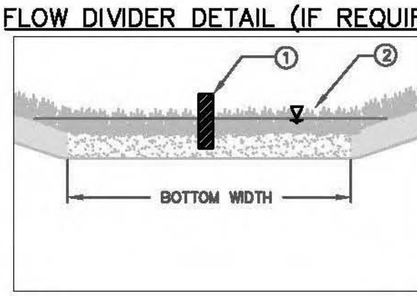

Section A – A'
(Not to Scale)




UNDERDRAIN DETAIL (IF REQUIRED)




FLOW DIVIDER DETAIL (IF REQUIRED)




NOTES:

① SWALE DIVIDER REQUIRED FOR BOTTOM WIDTHS > 7'. MINIMUM REQUIRED BOTTOM WIDTH IS 3'. MAXIMUM BOTTOM WIDTH WITH DIVIDER IS 14'.

② DEPTH OF FLOW FOR WATER QUALITY TREATMENT MUST NOT EXCEED 2 INCHES BELOW VEGETATION HEIGHT OR NOT GREATER THAN 6".

③ IF AN UNDERDRAIN IS REQUIRED, IT MUST CONSIST OF AN AT LEAST 6" DIAMETER PERFORATED PIPE IN COARSE AGGREGATE BED CONNECTED TO STORM DRAIN. GRAVEL BED MUST EXTEND 6" BELOW AND 12" TO THE SIDE AND TOP OF THE PIPE.

④ AMEND SOILS WITH 2" OF COMPOST TILLED INTO 8" OF NATIVE SOIL UNLESS NATIVE SOIL ORGANIC CONTENT > 10%.

⑤ INSTALL LEVEL FLOW SPREADER IF INFLOW IS CONCENTRATED

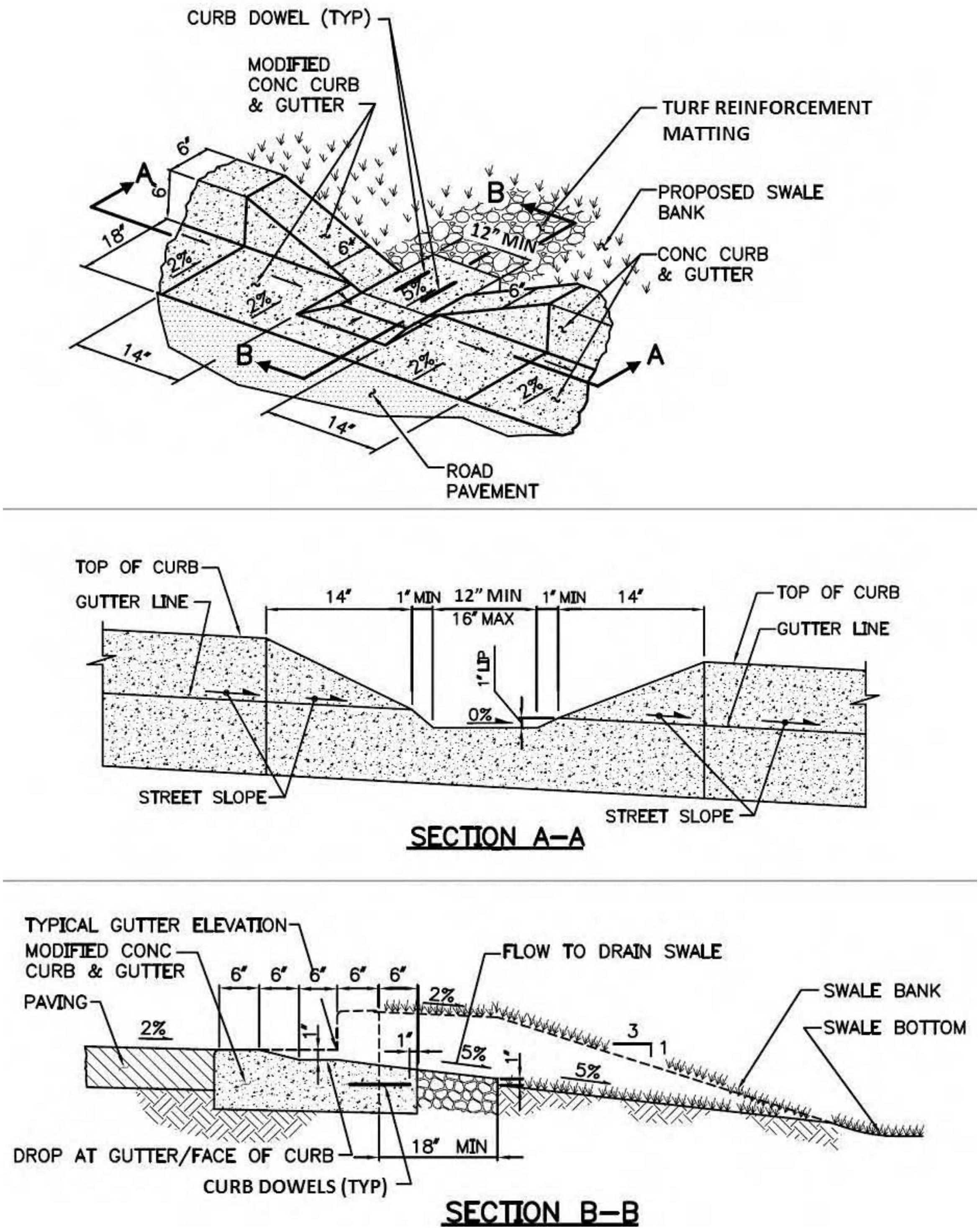
⑥ INSTALL ENERGY DISSIPATOR AT THE INLET OF VEGETATED SWALE.

⑦ SWALE LENGTH SHALL BE DETERMINED BASED ON A 9–MINUTE RESIDENCE TIME. IF SITE CONTSTRAITS REQUIRE SHORTER SWALE, MINIMUM RESIDENCE TIME SHALL BE 5 MINUTES. IF MULTIPLE INLETS, RESIDENCE TIME SHALL BE COMPUTED FOR AVERAGE FLOW DISTANCE

⑧ INSTALL APPROPRIATE OUTLET STRUCTURE TO ACCOMODATE PEAK DISCHARGE AND UNDERDRAIN (IF PRESENT).

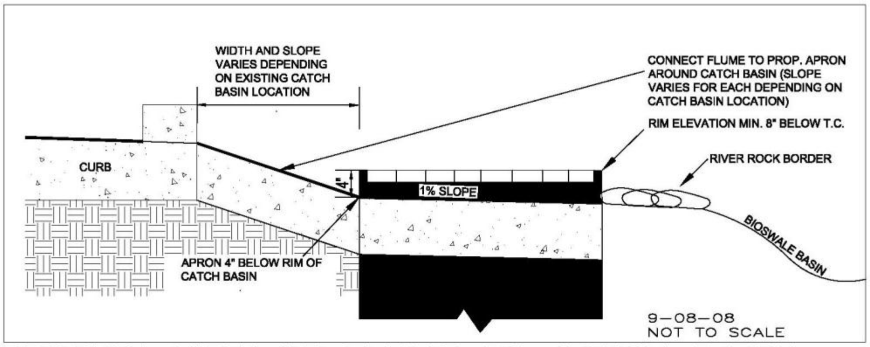
⑨ RECOMMENDED SIDE SLOPES ARE 4H:1V. SIDE SLOPES SHALL NOT BE STEEPER THAN 3H:1V

**Example Curb Cuts for Biofiltration Swales**
Source: Seattle Right-of-Way Manual, 2008

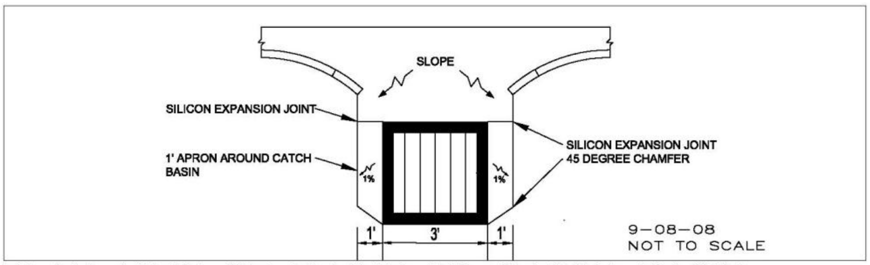


## Example Curb Cuts for Biofiltration Swales

Source: City of Florence, 2008

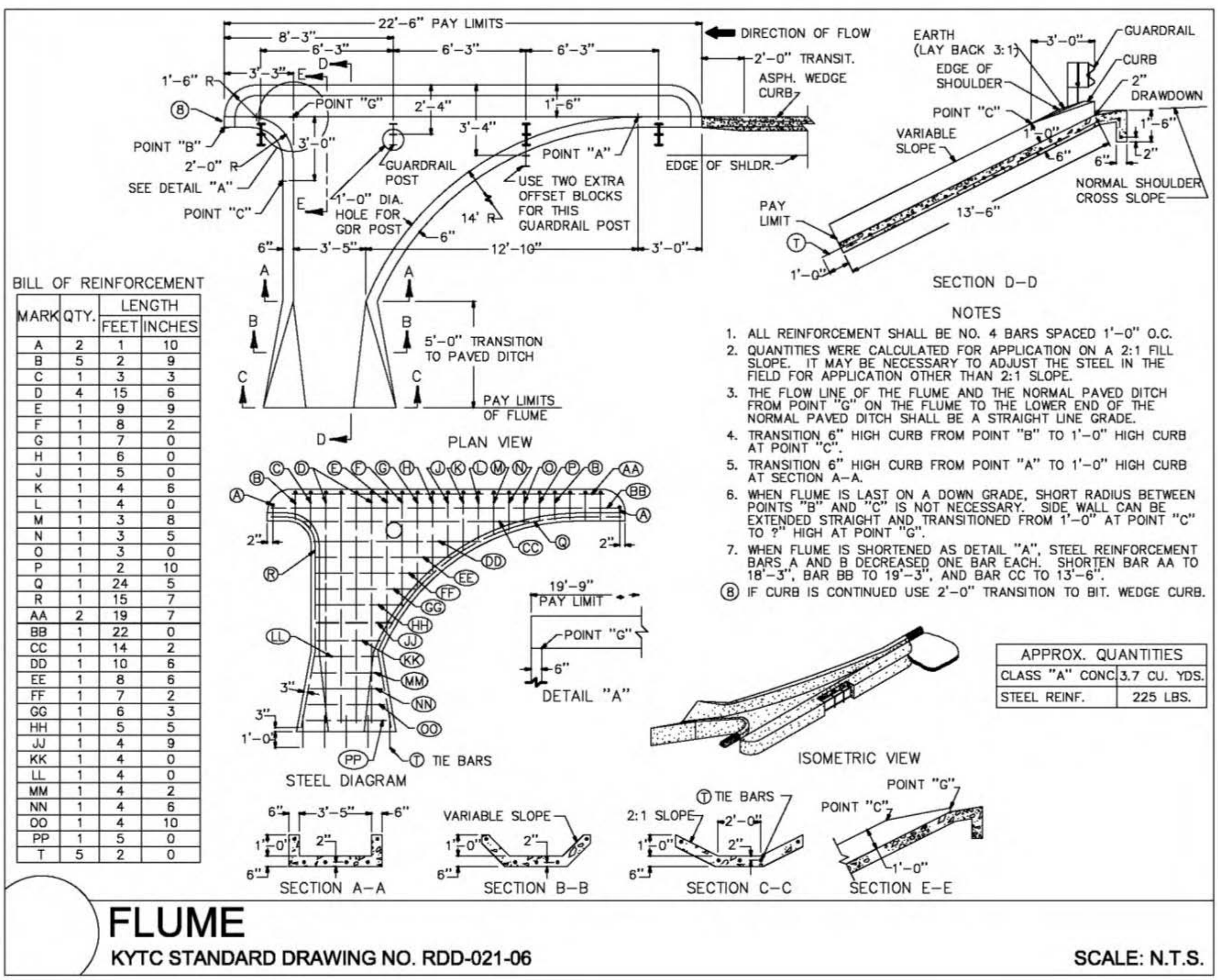



SECTION VIEW OF FLUME TO CATCH BASIN




PLAN VIEW OF FLUME TO CATCH BASIN

Source: Wetherington Blvd., City of Florence, 2008




BILL OF REINFORCEMENT

| MARK | QTY. | LENGTH | |
|---|---|---|---|
| | | FEET | INCHES |
| A | 2 | 1 | 10 |
| B | 5 | 2 | 9 |
| C | 1 | 3 | 3 |
| D | 4 | 15 | 6 |
| E | 1 | 9 | 9 |
| F | 1 | 8 | 2 |
| G | 1 | 7 | 0 |
| H | 1 | 6 | 0 |
| J | 1 | 5 | 0 |
| K | 1 | 4 | 6 |
| L | 1 | 4 | 0 |
| M | 1 | 3 | 8 |
| N | 1 | 3 | 5 |
| O | 1 | 3 | 0 |
| P | 1 | 2 | 10 |
| Q | 1 | 24 | 5 |
| R | 1 | 15 | 7 |
| AA | 2 | 19 | 7 |
| BB | 1 | 22 | 0 |
| CC | 1 | 14 | 2 |
| DD | 1 | 10 | 6 |
| EE | 1 | 8 | 6 |
| FF | 1 | 7 | 2 |
| GG | 1 | 6 | 3 |
| HH | 1 | 5 | 5 |
| JJ | 1 | 4 | 9 |
| KK | 1 | 4 | 0 |
| LL | 1 | 4 | 0 |
| MM | 1 | 4 | 2 |
| NN | 1 | 4 | 6 |
| OO | 1 | 4 | 10 |
| PP | 1 | 5 | 0 |
| T | 5 | 2 | 0 |

| APPROX. QUANTITIES | |
|---|---|
| CLASS "A" CONC. | 3.7 CU. YDS. |
| STEEL REINF. | 225 LBS. |

BIOFILTRATION SWALE

## MAINTENANCE

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Mow grass to maintain a height of 4-6 inches. |
| As needed (within 48 hours after every storm greater than 1 inch) | • Inspect and correct erosion problems and any damage to grass.<br>• Inspect swale inlet and outlet for blockages.<br>• Inspect check dams for erosion and stability. |
| As needed (infrequently) | • Remove sediment build-up, debris, and trash.<br>• Remove excess biomass or dethatch the swale surface if the thatch gets too dense.<br>• If stagnant water persists, regrade, rototill, and replant swale, modify outlet structure, or install underdrain. |
| Annually | • Plant alternative grass species if grass cover is not successfully established; re-seed bare or spotty patches. Use an erosion control mat.<br>• Inspect for and repair erosion channels (rills) alongside slopes.<br>• Inspect swale for cross-section and longitudinal slope uniformity and correct as needed. |

## ADDITIONAL SOURCES OF INFORMATION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# BIORETENTION / RAIN GARDEN

Structural Best Management Practice

BIORETENTION / RAIN GARDEN

http://www.water-research.net/urbanstormwaterbmp.htm

| PERFORMANCE | | | |
|---|---|---|---|
| H | Sediment | M | Bacteria |
| H | Metals | M | Trash and debris |
| M | Oil and grease | H | Volume Reduction |
| M | Nutrients | M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☑

Quality Control ☑

*Applications*

- Commercial and institutional
- Residential/subdivision
- Multi-family and mixed use
- Parking lots
- Road shoulders and medians
- Parks and golf courses

*Advantages*

✓ Suspended solids and particulate-bound pollutant removal
✓ Good removal of most dissolved pollutants
✓ Volume & peak flow reduction
Easily incorporated into site landscaping

*Limitations*

– Requires adequate vertical relief and proximity to storm drains if underdrain included
– Shallow ground water may not permit drawdown between storms
May leach nutrients immediately after installation

## DESCRIPTION

Bioretention areas and rain gardens are typically vegetated shallow depressions that provide storage, evapotranspiration, treatment and/or infiltration of captured storm water runoff. By filtering storm water through an engineered soil mix, bioretention areas and rain gardens can be designed to target a variety of pollutants. The primary storm water pollutant removal mechanisms in bioretention areas and rain gardens include filtration, shallow sedimentation, sorption and infiltration. Additional removal mechanisms include biochemical processes in the underlying engineered planting media such as adsorption and microbial transformations of dissolved pollutants. When properly incorporated into an overall site design, bioretention areas and rain gardens can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

There are two types of bioretention systems that can be used for storm water management, depending on the site needs and constraints:

- Bioretention without an Underdrain – These systems are designed to retain and infiltrate the water quality design volume from a site. These features can be implemented in areas where there are no hazards that would preclude infiltration (such as geotechnical concerns, shallow groundwater, or contaminant plumes or hazards) and where soil infiltration rates are relatively high (design infiltration rate >0.5 inch/hour, see design procedure section for instructions on how to calculate the design infiltration rate). Because these systems are built in moderately to highly infiltrating soils, they do not require the installation of an underdrain to draw down the ponded water within the required drawdown time.

- Bioretention with an Underdrain – Bioretention with an underdrain can be implemented in two ways. The first option involves the placement of the underdrain at the bottom of the facility. This is required when infiltration is hazardous due to geotechnical concerns, contaminant plumes, very high infiltration rates (>3.6 in/hr) with high pollutant generating source areas (e.g., gas stations), or other groundwater concerns. In some of these cases, the bioretention facility may need to be lined. This option can also be used when infiltration is simply not desired. The second option involves installing a raised underdrain in the facility. This is a good solution when infiltration rates are moderately low and infiltration is still desired. During a storm event, runoff will percolate down to the underlying granular drainage blanket and fill up the pore volume until the water level reaches the raised underdrain. The underdrain will then discharge the remaining volume that is not infiltrated, which allows for partial infiltration of all storms and complete treatment of the water quality design volume.

Cross sections and more information about how these systems differ are included in the sections below.

## SITE SUITABILITY CONSIDERATIONS

The following table summarizes general site suitability considerations for bioretention areas and rain gardens.

| SITE SUITABILITY CONSIDERATIONS FOR BIORENTENTION / RAIN GARDENS | |
|---|---|
| Tributary Area | < 5 acres (217,800 $ft^2$)[1] |
| Typical BMP area as percentage of tributary area (%) | < 5 percent |
| Proximity to steep sensitive slopes | A geotechnical investigation should be performed to determine feasibility and design constraints (e.g., necessity of underdrainage, minimum setbacks from crests and toes of slopes). |
| Depth to seasonally high groundwater table | < 5 ft, only bioretention with an underdrain systems can be used<br>> 5 ft, both systems can be used |
| Septic systems | Locate downgradient of primary and reserve drainfields |
| Hydrologic soil group | Any [2]:<br>• If measured infiltration rate < 2 in/hr, bioretention with an underdrain can be used<br>• If measured infiltration rate is > 2 in/hr, both system types can be used |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.
2 – If systems with underdrains are provided, site must have adequate relief between land surface and the storm water conveyance system to permit vertical percolation through the gravel drainage layer (open-graded base/sub-base) and underdrain to the storm water conveyance system.

The effectiveness of bioretention areas and rain gardens is directly related to the contributing land use, the size of the drainage area, the soil type, drainage area imperviousness, proposed vegetation, characteristics of engineered planting matrix, amount of storage provided, and the infiltration rate of the underlying soils. Natural low points in the topography are well-suited for bioretention and rain gardens, as are natural drainage courses, although infiltration capability may be reduced in these situations. Additional site suitability recommendations and potential limitations for bioretention areas and rain gardens are listed below.

- Placement – Placement of bioretention areas and rain gardens should take into account the location and function of other site features (buffers, undisturbed natural areas, etc.). Placement downstream of filter strips is recommended for roadside implementations. A licensed geotechnical engineer should be consulted in situations where steep slopes and structure foundations could potentially be impacted by infiltration from bioretention areas and rain gardens. If necessary a geotechnical report should be developed to document expected or measured infiltration rates of the in situ soils, the necessity of underdrains, and the minimum setbacks for the proposed features from toes/crests of slopes, areas of existing slope instability, and structure foundations. Bioretention areas and rain gardens should be located at least 10 feet from building foundations and should not be hydraulically connected to any structures or foundations.

- Soils –Where soils have moderately low to low permeability, a system with an underdrain must be used. Avoid constructing side slopes in fill material, which can be prone to erosion and/or structural damage by burrowing animals, if possible. If soils might be contaminated, bioretention with underdrain systems only may be used, and they must be lined.

- Shade – Areas with excessive shade may result in poor vegetative growth. For moderately shaded areas, shade tolerant plants and grasses shall be used.

BIORETENTION / RAIN GARDEN

## DESIGN CRITERIA

The following table summarizes the minimum design criteria for bioretention areas and rain gardens. Additional sizing criteria and design guidance are provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 for $V_{wq}$ calculations |
| Design media filtration rate | in/hr | Recommended 2 in/hr for bioretention media |
| Surface area | $ft^2$ | See Surface Area and Cross-Sectional Geometry section |
| Surface ponding depth | in | <8 inches |
| Required drain time | hr | 24 hours for ponded surface water (or maximum allowed for selected plant species)<br>48 hours for total device ponded water and storage layer(s) above invert of underdrain |
| Planting matrix thickness | ft | 2 - 3 feet minimum |
| Vegetation type | -- | Varies, must be water tolerant (see Vegetation section below) |
| Setbacks | ft | 5 feet minimum from structures and property lines along with additional constraints determined by geotechnical investigation |
| Side Slopes | | 3:1 max (gentler preferred) |
| Longitudinal Slope | | 1% or less |

### *Surface Area and Cross-Sectional Geometry*

- Surface area and effective storage depth must be adequate to capture, retain and treat the water quality design volume, $V_{wq}$. The effective storage depth is the surface storage plus the pore storage in the planting media.
- Planting matrix depth shall be 2 to 3 feet minimum. The intent is to provide a beneficial root zone for vegetation as well as contribute to storage capacity requirements for holding the design water quality volume.

### *Inflows and Energy Dissipation*

- Runoff can be directed into bioretention areas either as concentrated flows or as lateral sheet flow. Both are acceptable provided sufficient stabilization or energy dissipation and flow spreading is provided. If flow is to be directed into a bioretention area or a rain garden via curb cuts, provide a 2 to 3 inch drop at the interface of pavement and facility. Curb cuts should be at least 12 inches wide to prevent clogging and should be spaced appropriately to distribute the inflow as much as possible. The slope of the back curb should be 2 to 3% to guard against sediment aggradation and eventual blockage of inflow.
- Dispersed, low velocity flow across vegetated areas are the preferred inflow pattern; other inflows may include sheet flow across pavement or gravel.
- Concentrated flows shall be directed to a flow spreading trench around the edge of the bioretention area or other similar energy dissipation control.
- A flow spreader shall be used at the inlet so that the entrance velocity is quickly dissipated and the flow is uniformly distributed across the facility. Energy dissipation controls shall be constructed of sound materials such as stones, concrete, or proprietary devices that are rated to withstand the energy of the influent flows.

- Piped inflows, including roof downspouts should be directed to rocks, splash blocks or other equivalent energy dissipation / erosion control devices prior to discharging into the bioretention area or rain garden.

### *Overflow Structure*

- An overflow outlet structure shall be provided to drain runoff that exceeds the design surface ponding capacity of the facility. Overflow outlet structure may consist of a vertical PVC pipe, a gravel curtain, or equivalent structure connected to the underdrain (if included) or connected to the downstream storm drain system. If an overflow pipe is used, the overflow structure shall be 6 inches or greater in diameter. The inlet to the overflow structure shall be at least 6 inches above the surface of the planting media and shall be capped with a spider cap.

- If site conditions require the bioretention facility to be online, the overflow structure must be able to pass the flood control design flow rate ($Q_{fc}$) or an additional overflow structure (e.g., spillway) must be included to ensure flood flows can be safely routed back to the storm drain system without damaging the facility or causing flooding.

### *Underdrains*

- If underdrains are required, then they must be made of perforated or slotted, polyvinyl chloride (PVC) pipe conforming to ASTM D 3034 or equivalent or corrugated high density polyethylene (HDPE) pipe conforming to AASHTO 252M or equivalent. Underdrains shall slope at a minimum of 0.5 percent, and smooth and rigid PVC pipes shall be used as underdrains with slopes of less than 2 percent.

- The perforations or slots shall be sized to prevent the migration of the drain rock into the pipes, and shall be spaced such that the pipe has a minimum of 1 square inch of opening per lineal foot of pipe.

- The underdrain pipe must have a 6-inch minimum diameter, so it can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe must be placed at the terminal ends of the underdrain. The cleanout risers shall be plugged with a lockable well cap. It is recommended to keep the cap locked in areas prone to vandalism.

- The underdrain shall be bedded with 6 inches of drain rock and backfilled with a minimum of 6 inches of drain rock around the top and sides of the underdrain. The drain rock shall consist of clean, washed No. 57 stone, conforming to the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet, or an approved equal, that meets the gradation requirements listed in the table below.

| SIEVE SIZE | PERCENT PASSING |
|---|---|
| 1-$^{1}/_{2}$ inch | 100 |
| 1 inch | 95-100 |
| $^{1}/_{2}$ inch | 25-60 |
| US No. 4 | 0-10 |
| US No. 8 | 0-5 |

- The drain rock must be separated from the native soil layer below and to the sides with an approved non-woven geotextile fabric. The drain rock shall be separated from the planting media above with an approved non-woven geotextile fabric or with an appropriately graded granular filter. The graded granular filter should

BIORETENTION / RAIN GARDEN

consist of 2 to 4 inches of washed sand underlain with a minimum 2 inches of choking stone (washed No. 8 or No. 89 pea gravel). The non-woven geotextile filter fabric should not impede the infiltration rate of the planting media and should have a minimum flow rate of 50 gal/min/ft$^2$. Unless otherwise approved, the non-woven geotextile fabric shall conform to the Type II Fabric Geotextiles for Underdrains described in the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet. The minimum requirements for the non-woven geotextile filter fabric are listed in the table.

| GEOTEXTILE PROPERTY | VALUE | TEST METHOD |
|---|---|---|
| Grab Strength (lbs.) | 80 | ASTM D4632 |
| Sewn Seam Strength (lbs.) | 70 | ASTM D4632 |
| Puncture Strength (lbs.) | 25 | ASTM D4833 |
| Trapezoid Tear (lbs.) | 25 | ASTM D4533 |
| Apparent Opening Size US Std. Sieve | No. 50 | ASTM D4751 |
| Permeability (cm/s) | 0.010 | ASTM D4491 |
| UV Degration at 150 hrs. | 70% | ASTM D4355 |
| Flow Rate (gpm/ft$^2$) | 50 | ASTM D4491 |

- The underdrain pipe must drain freely to an acceptable discharge point.
- If no underdrains are present, an observation well extending at least 5 feet into native soil below the facility is recommended to assist with identifying drainage problems.
- For facilities that are not lined, the drain rock below the underdrain pipe should extend across the entire bottom of the facility to promote volume reductions.

### *Soils*

- The planting matrix of a rain garden or bioretention area must provide stability and adequate support for proposed vegetation. It must be highly permeable and high in organic content (e.g. loamy sand or sandy loam) and topped with a mulch layer 2-4 inches thick. The mulch layer should be shredded hardwood mulch or chips, aged a minimum of 12 months.
- Planting media design height shall be marked appropriately, such as a collar on the vertical riser (if installed), or with a stake inserted 2 feet into the planting media and notched to show bioretention surface level and ponding level.
- For bioretention areas with underdrains the media bed should consist of a minimum of 2 to 3 feet of bioretention soil mix above the underdrain. See Appendix B for guidance on bioretention soil mixes.
- For rain gardens, the site soil should be rototilled and amended prior to seeding. Unless the organic content is already greater than 10%, soils shall be amended with 2 inches of weed free and well-aged compost. The compost shall be mixed into the native soils to a depth of 6 inches to prevent soil layering and washout of compost. The compost will contain no sawdust, green or under-composted material, or any toxic or harmful substance. It shall contain no un-sterilized manure, which can lead to high levels of pathogen indictors (coliform bacteria) in the runoff. The compost shall be free of stones, stumps, roots or other similar objects larger than 3/4 inches.

### *Vegetation*

- Bioretention areas and rain gardens must be vegetated in order to provide adequate treatment of runoff via filtration. Vegetation, when chosen and maintained appropriately, also improves the aesthetics of a site.
- By incorporating into site landscaping, these facilities can be integrated into the overall site design without unnecessary loss of usable space.
- The selected plant materials shall be tolerant of summer drought, ponding fluctuations, and periodic saturated soil conditions (up to 24 hours) or other conditions specific to the BMP site (e.g., salt tolerance in areas with deicing operations).
- It is recommended that a diverse mix of trees[1], shrubs, and herbaceous groundcover species be incorporated to protect against facility failure due to disease or insect infestations of a single species.
- Plant rooting depths shall not damage underdrain if present. Slotted or perforated underdrain pipe should be more than 5 feet from tree locations (if space allows).
- Prohibited non-native plant species shall not be used. Refer to the Boone County Zoning Regulations (Landscaping section) for a list of prohibited plant species. Further information on invasive plant species in Kentucky can be found at the Early Detection & Distribution Mapping System (http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky).
- Tree plantings should allow enough light to pass to sustain a dense ground cover.



[1] Trees may require media depths of 3 feet or greater.

## DESIGN PROCEDURE

Rain gardens and bioretention with underdrain areas should be sized such that the ponded water drains within 24 hours and the entire facility above invert of underdrain completely drains within 48 hours. The intent is to replenish the facility storage capacity so that back to back storms can be adequately captured and treated. Simple sizing procedures for facilities with underdrains and facilities without underdrains are outlined below.

### ***Sizing for Facilities with Underdrains***

#### *Step 1: Design Volume*

The water quality design volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

#### *Step 2: Design Infiltration Rate*

If the facility includes the use of an underdrain, then the design infiltration rate is based on that of the bioretention planting matrix. For the planting matrix as specified in the Soils section above, a design infiltration rate of 2 in/hr should be assumed. The native soil infiltration rate should be determined using an in-situ percolation test measured at the elevation of the proposed bottom of the facility or at the depth of a limiting layer multiplied by a factor of safety of 0.25.

$$k_{media} = 2\ in/hr$$

$$k_{native} = (0.25)(k_{measured})$$

Where:

$k_{measured}$ = the infiltration rate determined from in-situ test

#### *Step 3: Facility Surface Area*

The required surface area can be calculated using the following equation:

$$A = \frac{12V_{wq}}{d_p + \eta \cdot d_{media}}$$

Where:

$A$ = required area of bioretention (ft$^2$)

$V_{wq}$ = water quality design volume (ft$^3$)

$d_p$ = design depth of ponding above bioretention area (8 inches or less)

$\eta$ = drainable porosity of the media (unitless); use 0.25 (This value is applicable to the bioretention soil mix specified in Appendix B. A different drainable porosity value may be approved with adequate documentation.)

$d_{media}$ = depth of planting media (min 24 inches)

#### *Step 4: Flow Capacity of Underdrain*

Underdrains must be designed so they drain water from the rock layer substantially faster than water enters from the media layer above. The design flow capacity of the underdrain pipe can be computed as:

$$Q_{und} = f_s \frac{k_{media}(A)}{(12)(3600)}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$f_s$ = factor of safety (use 5)

$k_{media}$ = design infiltration rate (use 2 in/hr)

$A$ = area of bioretention (ft$^2$)

### *Step 5: Number of Underdrain Pipes*

The diameter of a single pipe to convey the underdrain flow can be computed as:

$$D_s = 16\left(\frac{(Q_{und})(n)}{s^{0.5}}\right)^{3/8}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$D_s$ = single pipe diameter (in)

$n$ = Manning's roughness (use 0.011 for smooth pipe and 0.016 for corrugated pipe)

$s$ = pipe slope (recommended to be 0.005)

If more than one pipe is used, then this formula should be used to determine the sizing of the combination of pipes so that the sum of the flow rates of each pipe used is greater than or equal to $Q_{und}$.

### *Step 6: Facility Drawdown Time Above the Underdrain*

Compute the drawdown of the facility above the underdrain. The drawdown computed here represents the drawdown of the ponded area plus the drawdown of the media storage area. It does not include the drawdown of the gravel layer. Compute the drawdown using the following equation to ensure that complete drawdown occurs in no more than 48 hours:

$$T_{Tot} = \left[\frac{d_p + \eta_{media} d_{media}}{k_{media}}\right]$$

Where:

$T_{Tot}$ = total time to draw down both the ponded volume and the media volume (hours)

$d_p$ = design ponding depth (in) [max 8 inches]

$d_{media}$ = depth of planting media (in) [min 24 inches]

$k_{media}$ = media bed infiltration rate (in/hr); use 2 in/hr

$\eta_{media}$ = drainable porosity of the bioretention soil mix (unitless); use 0.25 (This value is applicable to the bioretention soil mix specified in Appendix B. A different drainable porosity value may be approved with adequate documentation.)

## Sizing for Facilities without Underdrains

### Step 1: Design Volume

The water quality design volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

### Step 2: Design Infiltration Rate

If the facility does not include an underdrain, then the design infiltration rate is based on that of the native soil as determined using an in-situ percolation test measured at the elevation of the proposed bottom of the facility or at the depth of a limiting layer multiplied by a factor of safety of 0.25.

$$k_{native} = (0.25)(k_{measured})$$

If $k_{measured}$ is less than 2 in/hr, then an underdrain is required.

### Step 3: Maximum Ponding Depth

Determine the maximum ponding depth based on the design infiltration rate and design drain time (24 hours maximum for ponded water to drain) as follows:

$$d_{\max} = (t)(k_{native})$$

Where:

$d_{max}$ = the maximum allowable depth of surface water (inches)
$k_{native}$ = design infiltration rate of underlying soil (in/hr)
$t$ = required drain time for ponded water (hrs) [24 hours max]

If $d_{max}$ is less than 8 inches then choose design depth less than or equal to $d_{max}$.

If $d_{max}$ is greater than 8 inches then choose design depth less than or equal to 8 inches.

### Step 4: Surface Area

The surface area computed here represents the surface area at the bottom of the slopes leading into the rain garden and not the area at the top of side slopes that may be considered as part of the facility. The drainable pore space volume of the media bed may be included as part of the design volume and the surface area should be calculated using the following equation:

$$A = \frac{12V_{wq}}{d_p + (\eta_{media})(d_{media})}$$

Where:

$A$ = required area of bioretention ($ft^2$)
$V_{wq}$ = water quality design volume ($ft^3$)
$d_p$ = depth of ponding above bioretention area (in)
$d_{media}$ = depth of planting media (in) [min 24 in]
$\eta_{media}$ = drainable porosity of the media bed (unitless); use 0.25 (This value is applicable to the bioretention soil mix specified in Appendix B. A different drainable porosity value may be approved with adequate documentation.)

### *Step 5: Entire Facility Drawdown Check*

Compute the drawdown of the entire facility. The drawdown computed here represents the drawdown of the ponded area plus the drawdown of the media storage area. Compute the drawdown using the following equation to ensure that complete drawdown occurs in no more than 48 hours. If it requires more than 48 hours, then either the media bed depth should be decreased or an underdrain should be installed.

$$T_{Tot} = \frac{d_p + \eta_{media} d_{media}}{k_{native}}$$

Where:

$T_{Tot}$ = total time to draw down both the ponded volume and the media volume (hours)

$k_{native}$ = infiltration rate of underlying soil (in/hr)

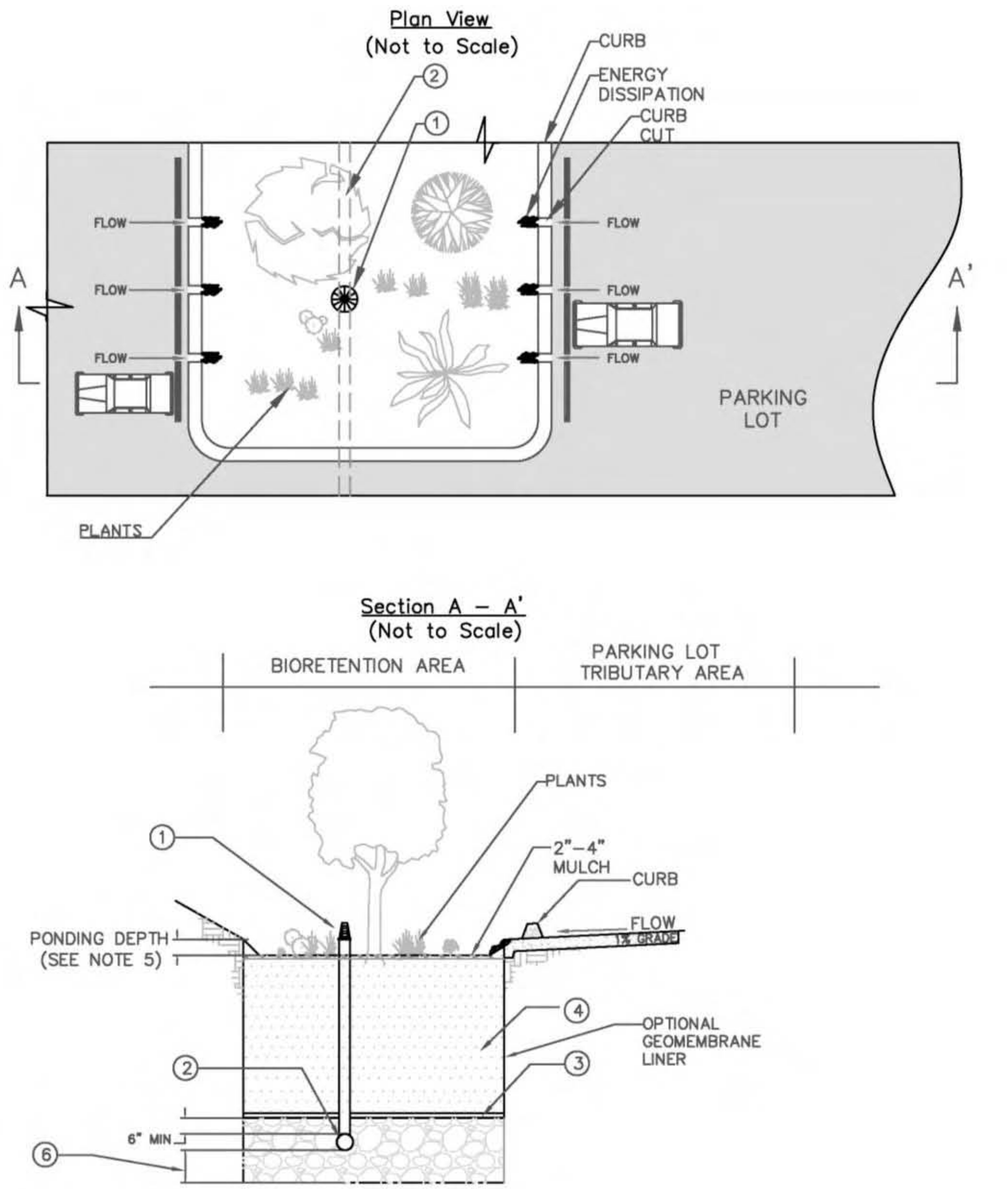
$d_p$ = design ponding depth (max 8 inches)

$d_{media}$ = depth of planting media (min 24 inches)

$\eta_{media}$ = drainable porosity of the media bed (unitless); use 0.25 (This value is applicable to the bioretention soil mix specified in Appendix B. A different drainable porosity value may be approved with adequate documentation.)

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of bioretention basins. Other designs are permissible if minimum design criteria are met.




NOTES:

1. OVERFLOW DEVICE: VERTICAL RISER OR EQUIVALENT.
2. PERFORATED 4" MIN PVC PIPE UNDERDRAIN. LOCATE ABOVE BASE OF GRAVEL LAYER TO FACILITATE INFILTRATION AND EXTEND GRAVEL LAYER ACROSS BOTTOM OF FACILITY AS NEEDED TO MAXIMIZE INFILTRATION STORAGE
3. OPTIONAL CHOKING GRAVEL LAYER. PLACE STRIP OF GEOTEXTILE OVER PIPE IMMEDIATELY BELOW CHOKING LAYER
4. 2' MIN PLANTING MIX; 3' PREFERRED. IF TREES, 3' MIN.
5. PONDING DEPTH 12" MAX
6. WHEN USING AN UNDERDRAIN, DEPTH OF GRAVEL SHOULD BE 6" MIN
NOTE: IF THERE IS NO INFILTRATION THE UNDERDRAIN MAY SIT AT THE BOTTOM OF THE GRAVEL LAYER

## MAINTENANCE

Bioretention areas and rain gardens require periodic plant, and planting matrix maintenance to ensure continued infiltration, storage and pollutant removal performance. A majority of the maintenance activities required are typical of landscaped areas.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Water plants as need until well established<br>• Maintain vegetation, prune and remove dead plant material.<br>• Remove any visual evidence of contamination from floatables<br>• Rake facility surface to facilitate infiltration of ponded runoff |
| As needed (within 48 hours after every storm greater than 1 inch) | • Inspect and correct erosion problems and any damage to vegetation.<br>• Inspect facility inlets and outlets for blockages.<br>• Clean and reset flow spreaders for optimum performance<br>• Remove sediment build-up, debris, and trash. |
| As needed (infrequently) | • Remove excess biomass if the vegetation gets too dense.<br>• If stagnant water persists, regrade, rototill, and re-vegetate, modify outlet structure, or install underdrain.<br>• Repair damage to flow control structures (inlet, outlet, and overflow)<br>• Clean out underdrain if present<br>• Replace planting matrix if infiltration capacity drops and re-vegetate<br>• Recommend documenting maintenance and taking photos before and after major maintenance. |
| Annually | • Plant alternative species if vegetation cover is not successfully established; re-seed bare or spotty patches.<br>• Replace mulch especially if high metal loadings are expected based on the land uses served.<br>• Inspect for and repair erosion channels (rills) alongside slopes.<br>• Snow shall not be dumped directly onto the bioretention/rain garden. |

BIORETENTION / RAIN GARDEN

## ADDITIONAL SOURCES OF INFORMATION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Boone County Planning Commission. Boone County Subdivision Regulations. 2010. http://www.boonecountyky.org/pc/2010SubdivisionRegs/2010SubRegs.pdf.

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Virginia Department of Conservation and Recreation. Virginia DCR Stormwater Design Specification No. 9: Bioretention. 2010. (refer to Appendix 9-A: Urban Bioretention).

# EXTENDED DETENTION BASIN

Structural Best Management Practice



Georgia Stormwater Management Manual
http://www.georgiastormwater.com/vol2/3-4-1.pdf

| PERFORMANCE | | | | |
|---|---|---|---|---|
| M | Sediment | | L | Bacteria |
| L | Metals | | H | Trash and debris |
| M | Oil and grease | | M | Volume Reduction |
| L | Nutrients | | H | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☑

Quality Control ☑

*Applications*

- Roads and highways
- Commercial developments
- Office building developments
- Multi-family developments

*Advantages*

✓ May be combined with flood control

✓ Efficient removal of sediments and associated pollutants

✓ Potentially significant volume mitigation

*Limitations*

– Performance very sensitive to basin configuration and outlet structure design

– Large footprint area

– Limited ability to remove dissolved pollutants

## DESCRIPTION

Extended detention (ED) basins (also known as dry ponds) are BMPs intended to provide: (1) water quality treatment, (2) volume reduction depending on site conditions, and (3) control of the peak flow rates and durations. ED basins do not have a permanent pool; they are designed to drain completely between storm events. Where soil conditions allow, they can provide significant volume reductions with infiltration. The side slopes, bottom, and forebay of ED basins are typically vegetated.

ED basins can be designed either on-line or off-line. If it is designed just for water quality treatment, it is recommended that the system be off-line from flood conveyance. For off-line basins, a flow diversion structure (i.e., flow splitter) should be used to divert the water quality design volume to the basin. On-line basins should be designed to pass the required flood event per SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations without damage to the basin, as well as to minimize re-entrainment of pollutants. For both types of basins, influent flows enter a sediment forebay where coarse solids are removed prior to flowing into the main cell of the basin, where finer sediment and associated pollutants settle as storm water is detained and slowly released through a controlled outlet structure. Low flows are often infiltrated within the basin if the basin is unlined. If standing water is a concern, a low flow drain can be installed.

## SITE SUITABILITY CONSIDERATIONS

Extended detention basins are large storage facilities that typically require 0.5 to 2.0 percent of the total tributary area. Tributary areas are generally larger than 10 acres. An ED basin can sometimes be retrofitted into existing flood control basins or integrated into the design of a park, athletic field, or other green space. Perforated risers, multiple orifice plate outlets, or similar multi-stage outlets are required for flood control retrofit applications. Multi-stage outlets ensure adequate detention time for small storms while still providing peak flow attenuation for the flood control design storm. Recreational multi-use facilities must be inspected after every storm and may require a greater maintenance frequency than dedicated water quality basins to ensure aesthetics and public safety are not compromised.

ED basins should not be placed on or near steep slopes. A geotechnical investigation is required if the basin is to be placed within 200 feet from a 15% or greater slope or a known landslide area. A liner may be required in such situations. A liner may also be required if the depth to the high water table is less than 5 feet from the bottom of the basin, the facility is within a 100 feet from a drinking water well, or in areas where a heightened threat of groundwater contamination may exist (e.g., industrial areas).

| SITE SUITABILITY CONSIDERATIONS FOR EXTENDED DETENTION BASINS | |
|---|---|
| Tributary Area[1] | > 10 acres (435,600 $ft^2$) |
| Typical BMP area as percentage of tributary area (%) | < 2 percent |
| Proximity to steep sensitive slopes | Basins placed on slopes greater than 15 percent or within 200 feet from a hazardous slope or landslide area require a geotechnical investigation |
| Depth to seasonally high groundwater table | < 5 feet, liner required<br>> 5 feet, no liner needed |
| Hydrologic soil group | Any |
| Distance to wells | 100 (private wells[2]) |
| Unsuitable locations | ED basins should not be placed within intermittent stream beds or in locations where an elevated threat of groundwater contamination may exist. Water levels should not be above those allowed by local zoning ordinances. |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.

2 – Public wells are governed by wellhead protection programs (A GIS layer showing protection program areas is available at http://kygisserver.ky.gov/geoportal/catalog/search/viewMetadataDetails.page?uuid=%7BEAE876B0-FBD0-4362-A7CA-75DA3B12BAA8%7D). Contact the Wellhead Protection Program Coordinator at the Kentucky Division of Water, Groundwater Branch for more information.

## DESIGN CRITERIA

ED basins have outlet structures that have been designed to detain the water quality design volume, $V_{wq}$, for 36 to 48 hours to allow sediment particles and associated pollutants to settle and be removed. To ensure adequate treatment of small storms while also providing quick recovery of available storage, the top half of the water quality design volume should drain twice as fast as the bottom half. For online basins that also provide flood control, the requirements of SD1's Storm Water Rules and Regulations and Boone County's Design Standards for Subdivision Regulations must also be met.

The following table summarizes the minimum design criteria for ED basins. Additional sizing criteria and design guidance are provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Flood control design discharge rate, $Q_{fc}$ | cfs | See SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. |
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 |
| Forebay basin size | $ft^3$ | 10-20% of total basin volume |
| Drawdown time | hr | Full $V_{wq}$ should drawdown in 36-48 hours |
| Freeboard (minimum) | in | 12 (offline)<br>24 (online) |
| Flow path length to width ratio | L:W | 3:1; can be achieved using internal berms |
| Side slope (maximum) | H:V | 3:1 (H:V) |
| Longitudinal slope | % | 1 (forebay) and 0-2 (main basin) |
| Low flow channel geometry | -- | See notes in Geometry and Size section |

EXTENDED DETENTION BASIN

### *Cross-Sectional Geometry and Size*

- The total basin volume shall be increased an additional 5% of the water quality design volume to account for sediment accumulation. If the basin is designed only for water quality treatment, then the basin volume would be 105% of the water quality design volume, $V_{wq}$. Freeboard shall be included above the total basin volume.

- The minimum flow-path length to width ratio at half basin height shall be a minimum of 3:1 (L:W) and can be achieved using internal berms or other means to prevent short-circuiting. Longer flow path lengths will improve fine sediment removal.

- The cross-sectional geometry across the width of the basin shall be approximately trapezoidal with a maximum side slope of 3:1 (H:V). Shallower side slopes are recommended if site conditions allow.

- All ED basins shall be free draining and a low flow channel shall be provided. A low flow channel is a narrow, shallow trench filled with pea gravel and encased with filter fabric that runs the length of the basin to drain dry weather flows. The low flow channel shall extend the entire length of the basin and shall have a positive-draining gradient flowing toward the outlet. The channel shall have a minimum depth of 6 inches and the width shall be sufficient to pass smaller storms but not be wider than 5% of the basin bottom width (typically about 1 foot wide). The low flow channel shall connect to a perforated pipe at the outlet structure. If a sand filter or planting media is provided beneath the ED basin for increased volume reduction, it may be designed to take the place of the low flow channel.

- The basin bottom shall have a 1% longitudinal slope (direction of flow) in the forebay, and may range from flat to 2% longitudinal slope in the main basin. The bottom of the basin shall slope 2% toward the center low flow channel.

### *Sediment Forebay*

As untreated storm water enters the ED basin, it passes through a sediment forebay for coarse solids removal. The forebay may be constructed using an internal berm constructed out of compacted and stabilized embankment material, riprap, gabion, stop logs, or other structurally sound material. If the berm is constructed out of earthen material, it should have a non-expansive clay core or otherwise be designed based on recommendations from a civil engineer licensed in Kentucky.

- The forebay should be 10-20% of the total basin volume.
- At the option of the designer, a gravity drain outlet from the forebay (4" minimum diameter) may be installed to allow complete drainage of the forebay. If used, the gravity drain must extend the entire width of the internal berm separating the forebay from the main basin, and an anti-seep collar shall be installed around the drain pipe.
- The forebay outlet shall be offset (horizontally) from the inflow streamline to address short-circuiting.
- Permanent steel post depth markers shall be placed in the forebay to define sediment removal limits at 50% of the forebay sediment storage depth.

### *Embankments and Side Slopes*

Embankments are earthen slopes or berms used for detaining or redirecting the flow of water. Basin embankments must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a civil engineer licensed in Kentucky) free of loose surface soil materials, roots, and other organic debris. Embankments should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. A slope no steeper than 4:1 is recommended for all slopes that will be mowed. Basin walls may be vertical retaining walls, provided: (a) a fence is provided along the top of the wall or further back from the basin edge, and (b) the retaining wall design is approved and stamped by a civil engineer licensed in Kentucky.

### *Outlet Structure and Drawdown Time*

A drawdown time of 36 to 48 hours shall be provided for the water quality design volume, $V_{wq}$. This drawdown time allows adequate time for pollutants to be settled and/or adsorbed. An outflow device shall be designed to release the bottom 50% of the water quality volume (half-full to empty) over 24 to 36 hours, and the top half (full to half-full) in 8 to 12 hours.

The outlet structure can be designed to achieve flow control for meeting the multiple objectives of water quality and flow attenuation. The outflow device (i.e., outlet pipe) shall be oversized (18 inch minimum diameter). There are two options that can be used for the outlet structure:

- Uniformly perforated riser structures, or
- Multiple orifice structures (orifice plate).

The outlet structure can be placed in the basin with a debris screen or housed in a standard manhole (see Appendix E and F for recommended outlet sizing methods and example structure designs, respectively). If a multiple orifice structure is used, an orifice restriction (if necessary) shall be used to limit orifice outflow to the maximum discharge rates allowable for achieving the desired water quality and flow control objectives. Orifice restriction plates shall be removable for emergency situations. A removable trash rack shall be provided at the outlet.

Note that a primary overflow (typically a riser pipe connected to the outlet works) shall be sized to pass flows larger than the water quality design storm (if the ED basin is sized only for water quality) or to pass flows larger than the peak flow rate of the maximum design storm to be detained in the basin. The primary overflow is intended to protect against overtopping or breaching of a basin embankment.

An anti-seep collar shall be installed for the outlet or any other pipe that penetrates the basin embankment.

### *Emergency Spillway*

Emergency overflow spillways are intended to control the location of basin overtopping and safely direct overflows back into the downstream conveyance system or other acceptable discharge point. Spillways should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

### *Energy Dissipation*

- Energy dissipation controls shall be constructed of sound material such as stones, concrete, or proprietary devices that are rated to withstand the energy of the influent flow, and shall be installed at the inlet to the sediment forebay. Flow velocity into the basin forebay shall be controlled such that it does not exceed 4 feet per second (ft/sec).
- Energy dissipation controls must also be used at the outlet/spillway from the detention basin unless the basin discharges to a storm drain or hardened channel.

### *Soils Considerations*

- ED basins can be used with almost all soils and geology. Geotechnical hazards and steep slopes must be subject to a geotechnical investigation approved and stamped by a civil engineer licensed in Kentucky prior to basin construction.
- If a liner is used, 1.5 to 2 feet of amended soil cover is recommended to protect the liner and ensure vegetation establishment.

### *Vegetation*

Vegetation within the ED basin shall provide erosion protection from wind and water and biotreatment of storm water. The following guidelines should be followed:

- The bottom and slopes of the ED basin shall be vegetated. A mix of erosion-resistant plant species that effectively bind the soil shall be used on the slopes and a diverse selection of plants that thrive under the specific site, climatic, and watering conditions shall be specified for the basin bottom. The basin bottom shall not be planted with trees, shrubs, or other large woody plants that may interfere with sediment removal activities. The basin shall be free of floating objects. Only native perennial grasses, forbs, or similar vegetation that can be replaced via seeding shall be used on the basin bottom.
- Landscaping outside of the basin is required for all ED basins and must adhere to the following criteria so as not to hinder maintenance operations:

- No trees or shrubs may be planted within 15 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, shall not be used within 50 feet of pipes or manmade structures. Weeping willow (Salix babylonica) shall not be planted in or near detention basins.
- Prohibited non-native plant species will not be permitted. Refer to the Boone County Zoning Regulations (Landscaping section) for a list of prohibited plant species. Further information on invasive plant species in Kentucky can be found at the Early Detection & Distribution Mapping System (http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky).

- A landscape professional should be consulted for project-specific planting recommendations, including recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth

### *Safety Considerations*

Safety is provided either by fencing the facility or by managing the contours of the basin to eliminate drop-offs and other hazards. Fencing shall meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. The design engineer must ensure that the final plans sufficiently protect maintenance crews and the general public from potential hazards associated with the basin design.

### *Maintenance Access*

- Maintenance access road(s) shall be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Manhole and catch basin lids must be in or at the edge of the access road.
- If it is not possible to access the basin bottom with equipment from outside the basin, a graded 16-foot wide access ramp near the basin outlet is recommended. Access is required for removal of sediment with a backhoe or loader and truck. The ramp must extend to the basin bottom to avoid damage to vegetation planted on the basin slope.

## DESIGN PROCEDURE

Extended detention basins should be sized to contain the total design volume plus 5% for sediment storage plus the freeboard requirements. Standard grading design should be implemented to estimate excavation and embankment fill quantities necessary while meeting the minimum design requirements described above.

### *Step 1: Calculate the Water Quality Design Volume*

The water quality design flow volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

### *Step 2: Calculate Preliminary Geometry Based on Site Constraints*

Determine the active volume of the forebay using the fractional volume ($FV_{fb}$) requirements for the forebay (10-20%) plus 5% for sediment accumulation. Similarly determine active volume of main cell using the fractional volume ($FV_{mc}$) requirements for the main basin (80-90%)

$$V_{fb} = 1.05 V_{wq} \frac{FV_{fb}}{100}$$

$$V_{mc} = 1.05 V_{wq} \frac{FV_{mc}}{100}$$

Where:
$V_{wq}$ = total water quality volume of extended detention ($ft^3$)
$FV_{fb}$ = fractional water quality volume of forebay (10 to 20%)
$FV_{mc}$ = fractional water quality volume of main cell (80 to 90%)
$V_{fb}$ = volume of forebay ($ft^3$)

Calculate surface area of forebay and main cell using average depths.

$$A_{fb} = \frac{V_{fb}}{D_{fb}}$$

$$A_{mc} = \frac{V_{mc}}{D_{mc}}$$

Where:
$A_{fb}$ = Active forebay surface area ($ft^2$)
$A_{mc}$ = Active main cell surface area ($ft^2$)
$V_{fb}$ = volume of forebay ($ft^3$)
$V_{mc}$ = volume of main cell ($ft^3$)
$D_{fb}$ = average depth of forebay (ft)
$D_{mc}$ = average depth of main cell (ft)

Select either a width or length for the facility based on site constraints and the space available and calculate remaining dimensions using the surface areas for the forebay and the main cell.

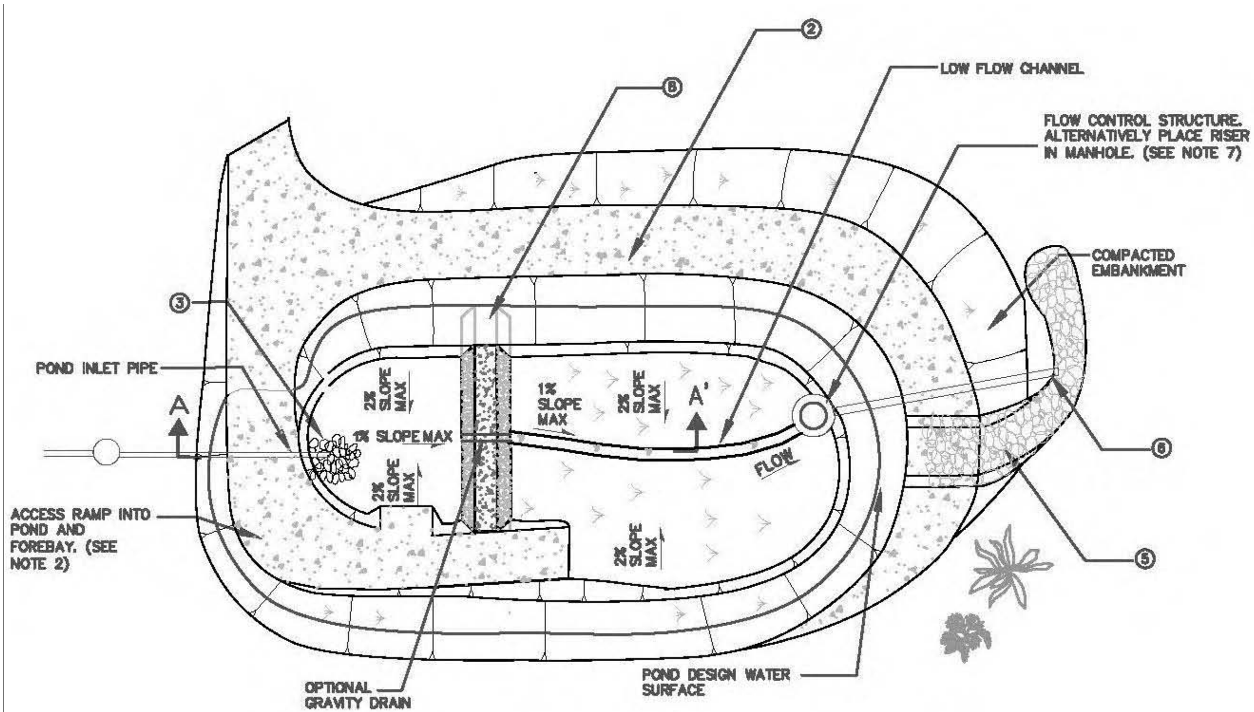
Calculate the non-active volumes and dimensions of the facility including berms, embankments and space needed for sediment storage. Add the non-active dimensions to the dimensions of the active forebay and main cell components to obtain the foot print dimensions of the facility.

### *Step 3: Select Flow Control Structures and Calculate Outlet Structure Dimensions*

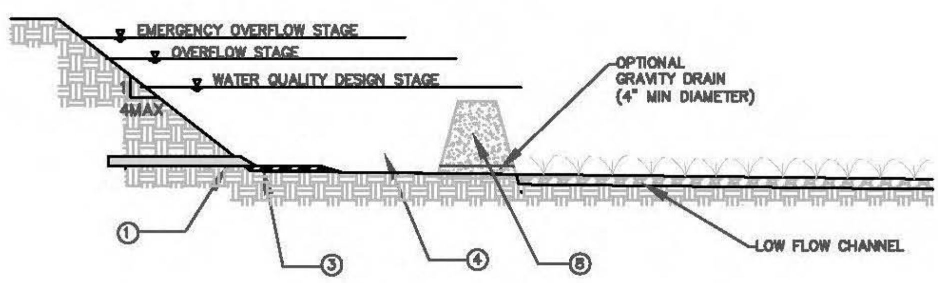
Provide adequate energy dissipation at inlets and size stilling basins as needed to prevent erosion. Recommended methods for sizing outlet structures for meeting the water quality drain time requirements and matching pre-development peak discharges are provided in Appendix E. Emergency spillways should be sized to convey the routed 100-yr design storm post-development peak flow rate. Refer to SD1's Storm Water Rules and Regulations or Boone County's Design Standards for Subdivision Regulation for acceptable methods for computing flood control design flows.

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of ED basins. Other designs are permissible if minimum design criteria are met.




Plan View
(Not to Scale)




Section A – A'
(Not to Scale)

NOTES:

1. INLET PIPE SHALL BE DESIGNED AND LOCATED TO MINIMIZE RE-SUSPENSION OF SEDIMENT.
2. MAINTENANCE RAMP SHOULD PROVIDE ACCESS TO BOTH THE FOREBAY AND MAIN BASIN.
3. RIP RAP APRON OR OTHER INLET ENERGY DISSIPATION SHALL BE PROVIDED SUCH THAT VELOCITIES IN THE FOREBAY ARE < 4 FT/S.
4. SEDIMENT FOREBAY SHOULD BE SIZED TO PROVIDE 20% OF THE TOTAL BASIN VOLUME.
5. EMERGENCY SPILLWAY MUST BE SIZED TO PASS 100-YEAR PEAK FLOW.
6. OUTLET PIPE. ENERGY DISSIPATION SHALL BE PROVIDED UNLESS DISCHARGE IS TO PIPE OR HARDENED CHANNEL.
7. OUTLET STRUCTURE SHOULD BE SIZED TO DRAIN WATER QUALITY VOLUME IN 36 HOURS. ALTERNATIVELY PLACE RISER STRUCTURE IN A MANHOLE.
8. CONSTRUCT BERM PER ENGINEER'S RECOMMENDATIONS.

EXTENDED DETENTION BASIN

## MAINTENANCE

Extended detention basins require periodic maintenance to maintain proper function. These maintenance activities focus on vegetation control, berm integrity, and removal of collected pollutants.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Remove trash and debris<br>• Remove evidence of visual contamination from floatables such as oil and grease<br>• Thin vegetation and mow as needed<br>• Eradicate noxious weeds |
| As needed (within 48 hours after every storm greater than 1 inch) | • Clean out sediment from inlets and outlets<br>• Stabilize slopes using erosion control measures (e.g. rock reinforcement, planting of grass, compaction)<br>• Verify pool drainage according to design specifications to avoid vector issues. |
| As needed (infrequently) | • Remove dead, diseased, or dying trees adjacent to the facility or those hindering maintenance activities.<br>• Replace any missing rock and soil at top of spillway.<br>• Remove forebay sediment when forebay capacity has been decreased by 50%. Remove sediment when six inches have accumulated across main basin bottom.<br>• Eliminate standing pools of water in low flow channel.<br>• Repair any damage of gate/fence. |
| Annually | • Verify basin embankments are not settling. Consult a civil engineer to determine the source of settling and whether corrective action is needed.<br>• Verify there are no discernible water flows through the basin embankments. Consult a civil engineer to inspect/correct if flow exists.<br>• Remove any trees or large shrubs growing on downstream side of berms to eliminate habitat for burrowing rodents.<br>• If a sand filter is included in the design of the ED basin, the surface should be inspected for signs of surface crusting and clogging. See the sand filter fact sheet for more information on the maintenance of sand filters. |

## ADDITIONAL SOURCES OF INFORMATION

AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001.

Boone County Planning Commision. Boone County Subdivision Regulations. 2010. http://www.boonecountyky.org/pc/2010SubdivisionRegs/2010SubRegs.pdf.

Cahill Associates, Inc. Pennsylvania Stormwater Best Management Practices Manual. 2006.

City of Portland, Oregon. Stormwater Management Manual. 2008. http://www.portlandonline.com/bes/index.cfm?c=47953&

Coastal Georgia Regional Development Center. Green Growth Guidelines. 2006.

Kentucky Division of Water. Design Criteria for Dams and Associated Structures (Engineering Memorandum No. 5). http://water.ky.gov/damsafety/Documents/WRmemo_5.pdf

Nashville, Tennessee. Stormwater Management Manual, Volume 4. 2009. http://www.nashville.gov/stormwater/regs/SwMgt_ManualVol04_2009.asp

Nevue Ngan Associated et al. Stormwater Management Handbook – Implementing Green Infrastructure in Northern Kentucky Communities. http://www.sd1.org/Resources.aspx?cid=3

Northern Virginia Planning District Commission (NVPDC) and Engineers and Surveyors Institute (ESI), 1992. Northern Virginia BMP Handbook: A Guide to Planning and Designing Best Management Practices in Northern Virginia. http://www.novaregion.org/index.aspx?nid=250

Sanitation District No. 1. Northern Kentucky Regional Storm Water Management Program: Rules and Regulations. 2011. Available at http://www.sd1.org/Resources.aspx?cid=9

Strecker, Eric and Klaus Rathfelder. Memo to Kentucky Sanitation District No. 1, Fort Wright, KY, 17 Nov. 2008.

Sanitation District No. 1. Northern Kentucky Regional Storm Water Management Program: Rules and Regulations. 2010. Available at http://www.sd1.org/Resources.aspx?cid=9

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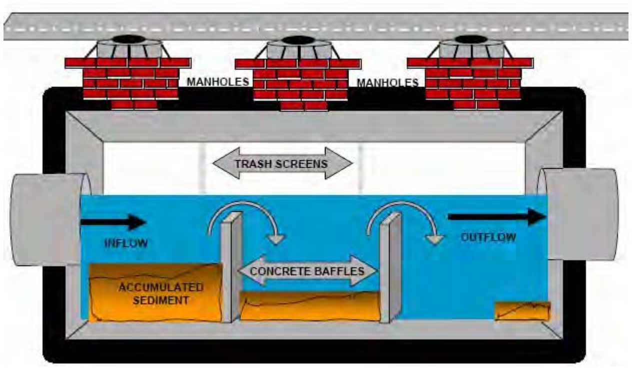
# GRAVITY SEPARATORS

Structural Best Management Practice






http://www.epa.gov/owm/mtb/baffle_boxes.pdf

| PERFORMANCE |
|---|
| Performance varies by device and manufacturer. |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☐

Quality Control ☑

*Applications*

- Roads, parking lots, gas stations
- Commercial and mixed use
- Industrial
- Residential
- Pretreatment for other BMPs

*Advantages*

✓ Small footprint required – can be placed below ground
✓ Ideal for retrofit situations
✓ Effective for gross solids removal
✓ Reduces maintenance requirements of downstream BMPs

*Limitations*

- Underground so “out of sight/out of mind”
- May become source of pollutants if not properly maintained
- Performance may need to be verified by an independent third party
- Not effective for dissolved pollutants

## DESCRIPTION

Gravity separators may consist of a variety of different types of structural devices designed to remove settleable solids, oil and grease, debris and floatables from storm water runoff through gravitational settling, skimming, and trapping of pollutants. Gravity separators primarily include baffle boxes and hydrodynamic separation devices.

Baffle Boxes are concrete or fiberglass structures containing a series of chambers separated by baffles. As storm water enters the box, suspended sediment settles and gets trapped in the chambers. Baffle boxes may contain trash screens or skimmers to capture larger materials, trash, and floatables.

Oil-water separators (also referred to as oil-grit separators) are a special type of baffle box specifically designed to remove gross pollutants including petroleum hydrocarbons, grease, sand, and grit. Interception of solid particles through settling, and floatation and skimming of oils and other floatables are fundamental processes occurring within an oil-water separator. There are two common designs for oil-water separators: the American Petroleum Institute (API) separator and the Coalescing Plate Separator (CPS). The API separator consists of three chambers divided by baffles and the first chamber acts as an equalization chamber where grit and larger solids settle and turbulent flow slows before entering the main separation chamber. The CPS, which is generally smaller than the API, uses a single baffle and a series of oil-attracting coalescing plates in the main chamber. In both types of devices, oil collects on the water surface where it can be skimmed off, absorbed to a floating media pad, or removed mechanically. Solids settle to the bottom and oil rises to the top, according to Newton's or Stokes' law depending on the flow regime. Larger oil-water separators contain a sludge scraper which continually removes the captured settled solids into a sludge pit. The oil is also removed by an oil skimming operation on the water surface.

Hydrodynamic Separation Devices (alternatively, swirl concentrators) are devices that remove trash, debris, and coarse sediment from incoming flows using screening, gravity settling, and centrifugal forces generated by forcing the influent into a circular motion. By taking advantage of centripetal forces caused by moving the water in circular fashion, it is possible to obtain significant removal of larger sediment particles and attached pollutants with less space as compared to wet vaults and other settling devices. Hydrodynamic devices were originally developed for combined sewer overflows (CSOs), where they were used primarily to remove coarse inorganic solids. Hydrodynamic separation has been adapted for storm water treatment by several manufacturers and is currently used to remove trash, debris, and other coarse solids down to sand-sized particles. Several types of hydrodynamic separation devices are also designed to remove floating oils and grease using sorbent media.

## SITE SUITABILITY AND PERFORMANCE CONSIDERATIONS

Site suitability is largely related to the type of treatment needed. In general, gravity separators are only effective at removing coarse sediment, trash and debris, and oil and grease. As such, these devices are primarily recommended for spill containment, pretreatment or for water quality retrofit of existing storm drains. Consequently, these devices are only recommended for pretreatment unless they have been specifically designed to treat the constituents of concern (see Design Criteria and Sizing section below).

Baffle boxes and hydrodynamic devices have a wide range of design elements (e.g., storage versus flow-through designs, inclusion of media filtration, etc.) that likely have significant effects on BMP performance; therefore, generalized performance data is not practical. Refer to data provided by the manufacturer and third-party sources to select a device that is effective at removing a particular suite of constituents of concern. The treatment effectiveness of specific proprietary devices must be provided by the manufacturer and shall be verified by independent third-party sources and data, or assessed by a water quality professional. One source of information providing independent information on proprietary devices (although a minority of them) is the EPA's Water Quality Protection Center – Verified Technologies web page (see reference section).

Hydrodynamic Separator; Contech Stormwater Solutions http://www.contech-cpi.com/Products/Stormwater-Management.aspx

## DESIGN CRITERIA AND SIZING

***Gravity separators may only be used as a standalone, primary treatment device if is specifically designed for addressing all of the constituents of concern.*** Only devices that have been approved by the State of Washington's Technology Assessment Protocol - Ecology (TAPE) Program[1] for addressing the constituents of concern are allowed to be used for primary treatment. For pretreatment, any device may be used provided it is properly sized and maintained according to the manufacturer's specifications. Additional general guidance on the design and sizing of these devices is provided below.

- BMP manufacturers are constantly updating and expanding their product lines, so refer to the latest device-specific design guidance and general guidelines for performance, sizing, operations and maintenance information.
- As a rule of thumb, baffle boxes should have footprint areas that are 2-4% of the tributary drainage area.
- While multiple sizes are possible and pre-fabricated tanks are available, typical baffle boxes are 10 to 15 feet long, 3 to 6 feet wide, and 6 to 8 feet high. Weir height is typically about 3 feet. Weirs are usually set at the same level as the pipe invert to minimize hydraulic losses. Manholes are set over each chamber to allow easy access for cleaning and maintenance. Manholes should be located within 15 feet of a paved surface to allow access by vacuum trucks for box maintenance.
- Generally these BMPs are designed as online flow-based BMPs, and therefore flow diverters are not needed; however, since individual performance varies with design, the manufacturer should be consulted for information on water quality performance at high flow rates before deciding whether or not to use a flow diverter with these BMPs. Sizing of proprietary devices is reduced to a simple process whereby a model can simply be selected from a table or a chart based on a few known quantities (tributary area, location, design flow rate, design volume, etc.). Some manufacturers either size the devices for potential clients or offer calculators on their websites that simplify the design process even further and lessens the possibility of using obsolete design information. For the latest sizing guidelines, refer to the manufacturer's website.

## DESIGN SCHEMATICS

Refer to manufacturers' websites for drawings of individual devices.

## MAINTENANCE

Refer to manufacturer instructions prior to performing any maintenance tasks.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Inspect devices 24 hours after first storms of the year and all storms greater than 0.5 inches.<br>• Remove gross solids that may clog inlet, etc. per manufacturer's recommended maintenance schedule. |
| As needed (infrequently) | • Refer to manufacturer's instructions for guidance on major sediment and solids removal, filter/sorbent media replacement, and structural repair schedule. |

[1] http://www.ecy.wa.gov/programs/wq/stormwater/newtech/technologies.html

## ADDITIONAL SOURCES OF INFORMATION

Refer to the table below for a partial list of available proprietary gravity separation devices. The mention of trade names or commercial products does not constitute endorsement or recommendation for use by SD1 or the City of Florence.



| DEVICE | MANUFACTURER | WEBSITE |
|---|---|---|
| ***Baffle Boxes:*** | | |
| Nutrient Separating Baffle Box® | Suntree Technologies, Inc. | www.suntreetech.com |
| Hydrasep® | Hydrasep, Inc. | www.hydrasep.com |
| Oil/Rainwater Runoff Separation | Facet International | www.facetinternational.com |
| ***Hydrodynamic Separators:*** | | |
| Rinker In-Line Stormceptor® | Rinker Materials™ | www.rinkerstormceptor.com |
| FloGard® Dual-Vortex Hydrodynamic Separator | KriStar Enterprises Inc. | www.kristar.com |
| Contech® CDS$^{a}$™ | Contech® Construction Products Inc. | www.contech-cpi.com |
| Contech® Vortechs™ | Contech® Construction Products Inc. | www.contech-cpi.com |
| Contech® Vortsentry™ HS | Contech® Construction Products Inc. | www.contech-cpi.com |
| BaySaver BaySeparator | Baysaver Technologies Inc. | www.baysaver.com |
| Aqua-Swirl® | Aquashield™ Inc. | www.aquashieldinc.com |

### *Other References*

AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001.

Center for Sustainable Design, Mississippi State University, 1999. *Water Related Best Management Practices in the Landscape*. Prepared for the Water Science Institute of the Natural Resource Conservation Service, U.S.D.A. [Online, Accessed June 2011] http://www.abe.msstate.edu/csd/NRCS-BMPs/water.html

England, G., 1999. "Baffle Boxes and Inlet Devices for Stormwater BMPs." http://terabook.net/baffle-boxes-and-inlet-devices-for-stormwater-bmp-s.html.

Khambhammettu, U., Pitt, R., Andoh, R and Clark, S, 2006. Performance of Upflow Filtration for Treating Stormwater. World Environmental & Water Resources Congress, ASCE/EWRI Omaha, Nebraska.

Lau, S.L., and Stenstrom, M.K. Best Management Practices to Reduce Pollution from Stormwater in Highly Urbanized Areas. Proceedings of the Water Environment Federation, WEFTEC 2002: Session 1 through Session 10, pp. 618-629(12).

Nashville, Tennessee. Stormwater Management Manual, Volume 4. 2009. http://www.nashville.gov/stormwater/regs/SwMgt_ManualVol04_2009.asp

U.S. EPA, 2001. Storm Water Technology Fact Sheet – Baffle Boxes. EPA 832-F-01-004. [Online, Accessed June 2011] http://www.epa.gov/owm/mtb/baffle_boxes.pdf

U.S. EPA. Water Quality Protection Center – Verified Technologies. http://www.epa.gov/nrmrl/std/etv/vt-wqp.html#SWSATD

# GREEN ROOF

Structural Best Management Practice

GREEN ROOF

Volume Control ☑

Quality Control ☑

Green roof on SD1 Administration Building(Photo: Sanitation District #1 of Northern Kentucky).

| PERFORMANCE* | | | |
|---|---|---|---|
| NA | Sediment | NA | Bacteria |
| NA | Metals | NA | Trash and debris |
| NA | Oil and grease | M | Volume Reduction |
| NA | Nutrients | M | Peak Flow Control |

NA – Not Applicable
H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.
* Green roofs are pimarily for volume control where water quality benefits are limited to the runoff volume reduced. Roof runoff is generally clean relative to other land uses, so reductions in pollutant concentrations may not occur.

*Applications*

- Commercial and institutional
- Residential
- Rooftops and decks above building structures
- Retrofit projects where building structure can support the additional weight

*Advantages*

✓ Volume & peak flow reduction
✓ May reduce required size of downstream BMPs
✓ Pollutant removal
✓ Potential decrease in cooling costs by insulating and shading buildings, and decreasing urban heat island effect
✓ Increased roof life; life-cycle costs comparable to traditional roof

*Limitations*

– Green roofs are heavier than conventional roofs and may require additional support
– Not applicable for roofs sloped greater than 25%
– High capital costs relative to traditional roof
– If leaks do occur, may be difficult to find

## DESCRIPTION

Green roofs, also known as ecoroofs, roof gardens, or vegetated roof covers, are roofing systems that consist of vegetative cover, growing media, a drainage layer, and a waterproof membrane. Green roofs reduce the effective imperviousness of buildings by storing direct rainfall within the planting media and allowing for subsequent evapotranspiration or slow release to local storm water conveyance systems. Because roof runoff is relatively clean, treatment effectiveness of green roofs is not comparable to other BMPs that treat runoff from a wide range of impervious surfaces that typically have higher pollutant concentrations.

There are two primary types of green roofs:

- Extensive – These green roofs are designed to be lighter (4 – 6 inches of soil and 20 – 30 lbs per $ft^2$) to minimize additional structural support necessary. They tend to be planted with various types of sedums. They are focused more on function and ease of implementation. Maintenance is minimal after the initial period of irrigation to establish the roof.

- Intensive – These green roofs are designed with greater soil depth so that a variety of plants (including grass, shrubs and trees) may be implemented. They require more maintenance and may be park-like in nature with playing fields. Weight per square foot varies considerably, but is commonly over 100 lbs/$ft^2$. Semi-intensive roofs are intermediate between intensive and extensive and often support lawns or other plant species not needing more than one foot of soil.

Tray type green roof systems are becoming more common, where the planting media and plants are contained in modular trays. Several manufacturers provide complete systems that include the trays, planting media, and underdrains.

Brown roofs are a variation of green roofs designed to maximize biodiversity. Brown roofs typically utilize natural soil and locally available substrates to create a protected biodiverse habitat for specific species of local flora and fauna. Rather than landscaping the roof during construction, plants are left to germinate and grow on their own in the native soils, thus the “brown” (i.e., initially unvegetated) designation. Hand-seeding may be implemented where self-colonization via airborne seeds is unlikely.

Blue roofs are a variation of green roofs designed specficially to store water. Typically blue roofs store rainfall to reduce the impact of storm water runoff or provide water for reuse. Blue roofs can be designed to store water on the open surface, beneath a porous media, or beneath a covered surface.

## SITE SUITABILITY CONSIDERATIONS

Green roof applicability is limited to rooftops and decks above building structures. Green roof siting should consider roof type and strength, as well as potential irrigation requirements until the roof vegetation is established. Other site suitability considerations are included below.

| SITE SUITABILITY CONSIDERATIONS FOR GREEN ROOFS | |
|---|---|
| Tributary Area | Equal to green roof area |
| Roof slope (%) | 0 – 25percent. Steeper roofs may need extra structural support to hold soil media and vegetation in place. |
| Roof Specification | Roof structure must be sufficient to support additional weight of green roof soil, vegetation, and stored water |

A summary of other green roof site suitability considerations include:

- Structural requirements – Buildings or structures proposed for green roofs must have the structural stability to support the additional weight of the green roof when filled with water. If green roofs are proposed for existing buildings, the structural integrity of the building should be evaluated by a structural engineer prior to installing a green roof.

- Development density – Green roofs are often good choices for developments where there may be challenges treating all runoff on-site. Green roofs can reduce runoff volume and flow rate, thus decreasing the required size of downstream BMPs.

- Shade – Areas with excessive shade may result in poor vegetative growth. For moderately shaded areas, shade tolerant sedums and grasses shall be used.

## DESIGN CRITERIA

The following table summarizes some relevant design criteria for green roofs. Additional design guidance is provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Soil depth range | in | 3-6 (extensive); 6 to 24 or more (intensive) |
| Saturated soil weight | $lb/ft^2$ | 15-35 (extensive); 60 – 200 (intensive) |
| Roof slope | % | 0-25 |

### *Geometry and Size*

Green roofs are self-mitigating (i.e., designed to treat direct rainfall) and they are not allowed to receive water from other impervious areas. Green roofs are generally intended to achieve moderate volume reduction and flow control.

### *Green Roof Structural Support*

- The roof must be able to support the additional weight of the soil, water, and vegetation.
- For retrofit projects a licensed structural engineer shall be consulted to determine the structural support present and what may need to be added to support the additional weight of 10 to 30 pounds per square foot.
- For new projects, the structural support concern shall be addressed during the design phase.

### *Green Roof Waterproofing*

- Waterproof roofing membrane is an integral part of a green roofing system. The waterproof membrane prevents the roof runoff from penetrating and damaging the roofing material. There are many materials available for this purpose; they come in various forms (i.e., rolls, sheets, liquid) and exhibit different characteristics (e.g., flexibility, strength, etc.). Depending on the type of membrane chosen, a root barrier may be required to prevent roots from compromising the integrity of the membrane.

### *Green Roof Soil Layer*

- Green Roof soil layers must have excellent drainage, not be too heavy when saturated, and be adequately fertile as a growing medium for plants. Many companies sell their own proprietary soil mixes. However, a simple mix of ¼ topsoil, ¼ compost, and the remainder pumice or perlite may be used for many applications. Other soil amendments may be substituted for the compost and the pumice or perlite. The soil mix used shall not contain any clay.
- A drainage layer below the soil layer is required to move the excess runoff off of the roof. There are numerous options including a gravel layer (that may require additional structural support), and many different styles and types of commercially available drain mats.

### *Drain*

- There must be a drain pipe (gutter) to convey runoff safely from the roof to another storm water runoff BMP, a pervious area, or the storm water conveyance system.

### *Vegetation*

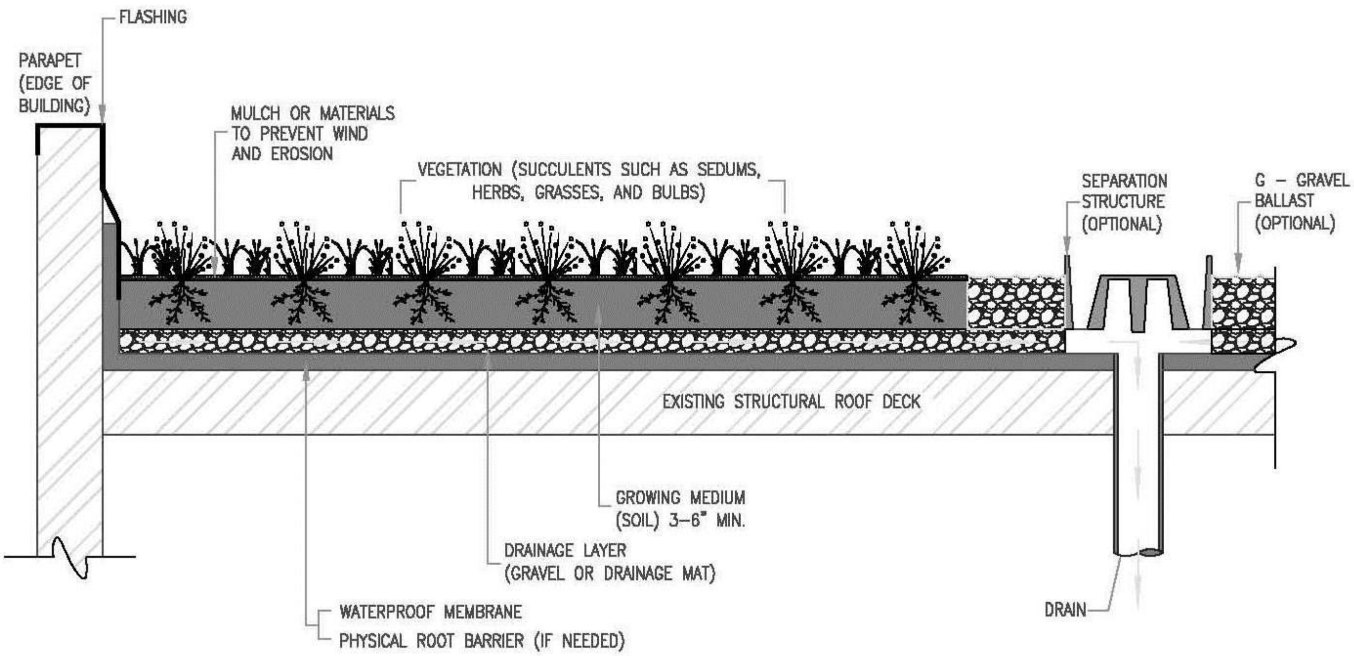
Vegetation of green roofs improves runoff water quality and increases transpiration. Plants have also been shown to increase the storage capacity of the roof, swelling with additional water and releasing it over time through evapotranspiration. Green roofs shall be about 90% vegetated with a mix of erosion-resistant plant species that effectively bind the soil and can withstand the extreme environment of rooftops. A diverse selection of herbs, succulents, and grasses that are drought tolerant, self-sustaining (perennial or self-sowing without need for fertilizers, herbicides, and or pesticides) is most effective. Plants selected shall also be low maintenance and able to withstand heat, cold, and high winds. Native or adapted sedum/succulent plants are preferred because they generally require less fertilizer, limited maintenance, and are more drought resistant than exotic plants. When appropriate, green roofs may be planted with larger plants; however, this is dependent of structural support and soil depth.

## DESIGN PROCEDURE

Because green roofs are self-sustaining and are not allowed to receive runoff from impervious areas, the size of the green roof is simply based on the size of the roof it is replacing. As such, there is no systematic sizing procedure for green roofs. However, green roofs can be used to reduce the size of downstream treatment facilities. As a rule of thumb, every square foot of green roof can be assumed to be 50% impervious when sizing downstream treatment facilities. This number can be slightly higher for intensive green roofs.

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of green roofs. Other designs are permissible if minimum design criteria are met.




Green Roof With Drainage Layer
(Not to Scale)

Source: City of Portland, Oregon.

GREEN ROOF

GREEN ROOF

## MAINTENANCE

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Irrigate until plants are established. Irrigation during dry periods may not be necessary if plants are properly selected.<br>• Maintain health of plants and remove any weeds or plants that interfere with the function of the green roof.<br>• Remove any visual contaminants and pollutants.<br>• Remove any trash and debris that has accumulated. |
| • As needed (within 48 hours after every storm greater than 1 inch) | • Inspect drain and remove any blocks or clogs.<br>• Inspect roofing membrane for signs of damage.<br>• Inspect roofing system for leaks.<br>• Inspect roof for signs of erosion or damage to vegetation. Identify causes and make corrective actions. |
| • As needed (infrequently) | • Clean and replace drainage layer.<br>• Re-vegetate areas where weeds and/or dead vegetation has been removed.<br>• Repair/replace waterproofing membrane. |

## ADDITIONAL SOURCES OF INFORMATION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# MEDIA BED FILTER

Structural Best Management Practice

MEDIA BED FILTER

http://www.aucklandcouncil.govt.nz/SiteCollectionDocuments/environment/sandfiltersconstructionguide.pdf

| PERFORMANCE | | | | |
|---|---|---|---|---|
| H | Sediment | | M | Bacteria |
| M | Metals | | H | Trash and debris |
| H | Oil and grease | | L | Volume Reduction |
| L | Nutrients | | M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☐

Quality Control ☑

*Applications*

- Ultra-urban roads and parking lots
- Commercial and industrial
- Multi-family residential

*Advantages*

✓ Efficient removal of particulate pollutants; active media may be used to target dissolved pollutants
✓ Good retrofit capability
✓ Good for highly impervious areas
✓ Applicable to small drainage areas

*Limitations*

- Potentially high maintenance burden
- Requires adequate vertical relief and proximity to storm drains
- Not recommended for runoff with high sediment content
- Usually little volume reduction due to underdrain

## DESCRIPTION

A media bed filter operates much like a bioretention area; however, instead of filtering storm water through planting soils, storm water is filtered through a constructed sand bed or other granular media with an underdrain system. Runoff enters the filter and spreads over the surface. As flows increase, water backs up on the surface of the filter where it is held until it can percolate through the sand. The treatment pathway is vertical (downward through the sand). High flows in excess of the design volume simply spill out over the top of the pool or over a designed spillway. Water that has percolated through the sand is collected via a perforated underdrain system before being conveyed to another BMP type, storm water conveyance system, or being daylighted and dispersed over a pervious area. As storm water passes through the media bed, pollutants are trapped in the small pore spaces between sand grains or adsorbed to the media surface.

Because they have few site constraints besides head requirements, sand filters can be used on development sites where the use of other structural controls may be precluded. However, sand filter systems can be relatively expensive to construct, install, and maintain in comparison to rain gardens and bioretention areas.

There are three general sand filter designs:

1. **Surface Sand Filter** – The surface sand filter is a ground-level open air structure that consists of pretreatment (e.g., vegetated BMP, proprietary device, or sediment forebay) and a filter bed chamber with perforated drain pipe under the filter bed. This system can treat drainage areas up to 10 acres in size and is typically off-line. Surface sand filters can be designed as an excavation with earth embankments or as a concrete or block structure.

2. **Perimeter Sand Filter** – The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation (pretreatment) chamber and a sand bed filter with underdrain. Runoff flows into the sedimentation chamber through a series of inlet grates located along the top of the control.

3. **Underground Sand Filter** – The underground sand filter is primarily for extremely space limited and high density areas and consists of a three-chamber system. The initial chamber is a sedimentation (pretreatment) chamber that temporarily stores runoff and utilizes a wet pool to capture sediment. The sedimentation chamber is connected to the sand filter chamber by a submerged wall that protects the filter bed from oil and trash. Perforated drain pipes under the sand filter bed extend into the third chamber that collects filtered runoff. Flows beyond the filter capacity are diverted through an overflow weir.

## SITE SUITABILITY CONSIDERATIONS

Media bed filter systems are generally applied to drainage areas with a high percentage of impervious surfaces. If the filter receives runoff from pervious areas, these areas should be well vegetated and stabilized. Pervious areas with high clay/silt sediment loads must not use sand filters without adequate pretreatment because the sediment causes clogging and failure of the filter bed.

The following table summarizes general site suitability considerations for media filters.

| SITE SUITABILITY CONSIDERATIONS FOR MEDIA BED FILTERS | |
|---|---|
| Tributary Area | < 10 acres (435,600 $ft^2$) for surface sand filter, < 2 acres for perimeter sand filter, and < 1 acres for underground sand filter |
| Typical BMP area as percentage of tributary area (including settling chamber) | 2 to 4 percent |
| Proximity to steep sensitive slopes | If system is fully contained and includes a liner, underdrain system, and overflow to a storm drain system, then slopes can exceed 15 percent. |
| Depth to seasonally high groundwater table | > 2 ft with underdrains<br>> 10 ft without underdrains |
| Hydrologic soil group | Any |
| Unsuitable locations | Media filters should not be placed within 200 feet of drinking water wells if media filter does not have an underlying impermeable liner or is not contained within a concrete vault. |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.

## DESIGN CRITERIA

Sand filters are volume-based BMPs intended, primarily, for treating the water quality design volume, $V_{wq}$. In most cases, sand filters are enclosed concrete or block structures with underdrains; therefore, only minimal volume reduction occurs via evaporation as storm water percolates through the filter to the underdrain. A hybrid combining sand filters and dry extended detention basins can be designed with or without underdrains and utilize the sand filter as a filtration and storage layer allowing storm water to be detained and filtered (if underdrains are included) or, if site conditions allow, infiltrated into the subsoil (if underdrains are omitted). In this hybrid case, volume reduction can be achieved. The following table summarizes the minimum design criteria for media filters. Additional sizing criteria and design guidance are provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 |
| Forebay basin size | $ft^3$ | 20 - 25% of total basin volume if no other pretreatment is used |
| Drawdown time for $V_{wq}$ | hr | 24 |
| Freeboard (minimum) | in | 12 |
| Flow path length to width ratio | L:W | 1.5:1 |
| Longitudinal slope | % | 0 – 2 |
| Filter bed depth | in | 24 inches, 36 inches preferred |
| Max ponding depth above filter bed | ft | 3 |
| Hydraulic conductivity of sand $k_{media}$ | in/hr | 1.5 (or lab measured values reduced by a factor of 4) |
| Underdrains | -- | 6” minimum diameter, 0.5% minimum slope |

### *Pretreatment*

Pretreatment must be provided for sand filters in order to reduce the sediment load entering the filter so that the potential for clogging is minimized. Pretreatment refers to design features that provide settling of large particles before runoff reaches a management practice, easing the long-term maintenance burden. Example pretreatment BMPs include vegetated swales, filter strips, proprietary devices, or sedimentation forebays.

### *Sizing and Geometry*

- Sand filters shall be sized to capture and filter the water quality design volume, $V_{wq}$ (see Chapter 3).
- Sand filters may be designed in any geometric configuration, but rectangular with a 1.5:1 length to width ratio or greater is preferred.
- Filter bed depth must be at least 24 inches and preferably 36 inches.
- Depth of water storage over the filter bed shall be 3 feet maximum.
- Sand filters shall be placed off-line to prevent scouring of the filter bed by high flows. The overflow structure must be designed to pass the water quality design storm.

### *Sand Specification*

Ideally the effective diameter of the sand, $d_{10}$, should be just small enough to ensure a good quality effluent while preventing penetration of storm water particles to such a depth that they cannot be removed by surface scraping (~2-3 inches). This effective diameter usually lies in the range 0.20-0.35 mm. In addition, the coefficient of uniformity, $C_u = d_{60}/d_{10}$, shall be less than 3.

The sand in a filter shall consist of a medium sand with very little fines meeting ASTM C 33 size gradation (by weight) or equivalent as given in the table below.

| U.S. SIEVE SIZE | PERCENT PASSING |
|---|---|
| 3/8 inch | 100 |
| U.S. No. 4 | 95 to 100 |
| U.S. No. 8 | 80 to 100 |
| U.S. No. 16 | 50 to 85 |
| U.S. No. 30 | 25 to 60 |
| U.S. No 50 | 5 to 30 |
| U.S. No. 100 | 0 to 10 |
| U.S. No. 200 | 0 to 3 |



### *Alternative Media*

Although sand is the most common media for use in filters, alternative media with desirable hydraulic or physiochemical properties may also be used, such as zeolite, granular activated carbon (GAC), peat and sand mixtures. Most of these function by increasing the specific surface area, organic content, and cation exchange capacity of the media bed, and are therefore more effective than inert sand filters at removing dissolved constituents such as organic compounds, nutrients, and dissolved metals.

### *Underdrains*

- If underdrains are required, then they must be made of perforated or slotted, polyvinyl chloride (PVC) pipe conforming to ASTM D 3034 or equivalent or corrugated high density polyethylene (HDPE) pipe conforming to AASHTO 252M or equivalent. Underdrains shall slope at a minimum of 0.5 percent, and smooth and rigid PVC pipes shall be used as underdrains with slopes of less than 2 percent.
- The perforations or slots shall be sized to prevent the migration of the drain rock into the pipes, and shall be spaced such that the pipe has a minimum of 1 square inch of opening per lineal foot of pipe.
- All underdrain pipes and connectors must have a 6-inch minimum diameter, so they can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe must be placed at the terminal ends of all pipes and extend to the surface of the filter. A valve box shall be provided for access to the cleanouts and the cleanout assembly must be water tight to prevent short circuiting of the sand filter.
- The underdrain shall be bedded with 6 inches of drain rock and backfilled with a minimum of 6 inches of drain rock around the top and sides of the underdrain. The drain rock shall consist of clean, washed No. 57 stone, conforming to the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet, or an approved equal, that meets the gradation requirements listed in the table below.

| SIEVE SIZE | PERCENT PASSING |
|---|---|
| 1-$^1/_2$ inch | 100 |
| 1 inch | 95-100 |
| $^1/_2$ inch | 25-60 |
| US No. 4 | 0-10 |
| US No. 8 | 0-5 |

- The drain rock must be separated from the native soil layer below and to the sides with an approved non-woven geotextile fabric. The drain rock shall be separated from the media filter above with an approved non-woven geotextile fabric or with an appropriately graded granular filter. The graded granular filter should consist of a minimum 2 inches of choking stone (washed No. 8 or No. 89 pea gravel). The non-woven geotextile filter fabric should not impede the infiltration rate of the planting media and should have a minimum flow rate of 50 gal/min/ft$^2$. Unless otherwise approved, the non-woven geotextile fabric shall conform to the Type II Fabric Geotextiles for Underdrains described in the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet. The minimum requirements for the non-woven geotextile filter fabric are listed in the table below.

| GEOTEXTILE PROPERTY | VALUE | TEST METHOD |
|---|---|---|
| Grab Strength (lbs.) | 80 | ASTM D4632 |
| Sewn Seam Strength (lbs.) | 70 | ASTM D4632 |
| Puncture Strength (lbs.) | 25 | ASTM D4833 |
| Trapezoid Tear (lbs.) | 25 | ASTM D4533 |
| Apparent Opening Size US Std. Sieve | No. 50 | ASTM D4751 |
| Permeability (cm/s) | 0.010 | ASTM D4491 |
| UV Degration at 150 hrs. | 70% | ASTM D4355 |
| Flow Rate (gpm/ft$^2$) | 50 | ASTM D4491 |

- Several underdrain systems can be used in a sand filter design:
    - Central underdrain collection pipe with lateral collection pipes in a gravel backfill or drain rock bed.
    - Longitudinal pipes in a gravel backfill or drain rock bed, with a collection pipe at the outfall.
    - Small sand filters may utilize a single underdrain pipe in a gravel backfill or drain rock bed.
- The maximum perpendicular distance between any two lateral collection pipes or from the edge of the filter and the collection pipes shall be 9 feet.
- The underdrain pipe must drain freely to an acceptable discharge point.

### *Flow Spreading*

- A flow spreader shall be installed at the inlet along one side of the filter to evenly distribute incoming runoff across the filter and to prevent erosion of the filter surface.
    - If the sand filter is curved or an irregular shape, a flow spreader shall be provided for a minimum of 20 percent of the filter perimeter.
    - If the length-to-width ratio of the filter is 2:1 or greater, a flow spreader must be located on the longer side and for a minimum length of 20 percent of the facility perimeter.

- In other situations, use good engineering judgment in positioning the spreader.

- Erosion protection shall be provided along the first foot of the sand bed adjacent to the flow spreader. Geotextile weighted with sand bags at 15-foot intervals may be used. Quarry spalls (small rock) may also be used.



### *Emergency Overflow Structure*

Sand filters shall be placed off-line, but an emergency overflow must still be provided in the event the filter becomes clogged. The overflow structure must be able to safely convey flows from the water quality design storm to the downstream storm water conveyance system or other acceptable discharge point. The invert of the overflow structure must be at the routed water quality design storm water surface elevation in the facility. The top of facility shall be 1 foot above this elevation to provide 1 foot of freeboard between the routed water quality design storm water surface elevation and the top of facility.

### *Containment Structure*

Media bed filters may be contained using earthen berms or reinforced concrete structures that are either pre-cast or cast-in-place. If earthen containment is used, basin embankments must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed civil engineer in Kentucky) free of loose surface soil materials, roots, and other organic debris. Embankments should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

### *Safety Considerations*

Safety is provided either by fencing the facility or by managing the contours of the facility to eliminate drop-offs and other hazards. Fencing shall meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. The design engineer must ensure that the final plans sufficiently protect maintenance crews and the general public from potential hazards associated with the media filter design.

### *Maintenance Access*

- Safe maintenance access shall be provided to the media bed surface and underdrain clean-out risers.
- For large facilities where the entire media bed cannot be accessed from outside the basin, an access ramp extending to the basin bottom is required for removal of sediment with a backhoe or loader and truck.

## DESIGN PROCEDURE

A media bed filter is a volume-sized BMP designed with two parts: (1) a temporary storage reservoir to store runoff, and (2) a filter bed through which the stored runoff must percolate. Usually the storage reservoir is simply placed directly above the filter, and the floor of the reservoir pond is the top of the filter bed. For this case, the storage volume also determines the hydraulic head over the filter surface, which increases the rate of flow through the media bed.

The method below applies to all media bed filter types. The primary differences are in the configuration and location of the media bed. Pretreatment for underground and perimeter sand filters is typically a forebay/sedimentation chamber whereas multiple options are available for surface sand filters. If a forebay is used for pretreatment, the storage volume below the depth of overflow to the media bed surface should equal to 10 - 20% of the water quality design volume and the flow length-to-width ratio is recommended to be 2:1 or greater unless baffles or inclined plate settlers are used.

### *Step 1: Calculate storage depth*

Determine the maximum water storage depth, $d_{\max}$, above the sand filter. This depth is defined as the depth at which water begins to overflow the temporary storage reservoir, and it depends on the site topography and hydraulic constraints. The depth is chosen by the designer, but shall be 6 feet or less.

$$d_{\max} = \frac{(k_{media})(t)}{12}$$

Where:

$d_{max}$ = the maximum depth of surface water that can be infiltrated within the required drain time(ft)

$k_{media}$ = design infiltration rate of the media bed (in/hr); [use 1.5 in/hr for coarse sand to account for long-term average performance; otherwise use laboratory measured values reduced by a factor of 4]

$t$ = required drain time for ponded water (hrs); [24 hrs max]

Choose a design depth of ponding, $d_p$, such that:

$$d_p \le d_{\max}$$

### *Step 2: Calculate the design volume*

Determine water quality design volume, $V_{wq}$ (see Chapter 3 for details).

### *Step 3: Calculate the sand filter area*

The area of a sand filter may be sized using a volumetric approach and a Darcy's law-based approach. Computations should be made using both methods and the larger of the two areas should be selected for design.

Method 1: Volumetric approach. Determine the sand filter area, $A_1$, using the following equation:

$$A_1 = \frac{V_{wq}}{d_p + (\eta_{media})(d_{media})}$$

Where:

$A_1$ = area of the filter media bed using volumetric approach ($ft^2$)

$V_{wq}$ = water quality design volume ($ft^3$)
$d_p$ = depth of ponding above the media bed (ft); [max of 6 ft]
$d_{media}$ = depth of filter media (in); [min 2 ft]
$\eta_{media}$ = drainable porosity of the media bed (unitless); [use 0.32 for sand or laboratory measured drainable porosity]

Method 2: Darcy's Law approach. Determine the sand filter area, $A_2$, using the following equation:

$$A_2 = \frac{(l)(V_{wq})}{(t)\left(\frac{k_{media}}{12}\right)(h+l)}$$

Where:

$A_2$ = area of the filter media bed using Darcy's law-based approach ($ft^2$)
$V_{wq}$ = water quality design volume ($ft^3$)
$k_{media}$ = design infiltration rate of the media bed (in/hour)
$l$ = depth of media bed (ft)
$h$ = average depth of water above the filter (ft); use $d_p/2$ with $d_p$ determined in Step 1
$t$ = required drawdown time for ponded water (hours); [use 24 hours]

Choose a design area of the media bed, $A_{media}$, such that:

$$A_{media} = \max(A_1, A_2)$$

### *Step 4: Flow Capacity of Underdrain*

Underdrains must be designed so they drain water from the rock layer substantially faster than water enters from the media layer above. The design flow capacity of the underdrain pipe can be computed as:

$$Q_{und} = f_s \frac{k_{media}(A_{media})}{(12)(3600)}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)
$f_s$ = factor of safety [use 5]
$k_{media}$ = design infiltration rate [use 1.5 in/hr]
$A_{media}$ = area of media filter ($ft^2$)

### *Step 5: Underdrain Pipe Diameter*

The diameter of a single pipe to convey the underdrain flow can be computed as:

$$D_s = 16\left(\frac{(Q_{und})(n)}{s^{0.5}}\right)^{\frac{3}{8}}$$

MEDIA BED FILTER

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$D_s$ = single pipe diameter (in)

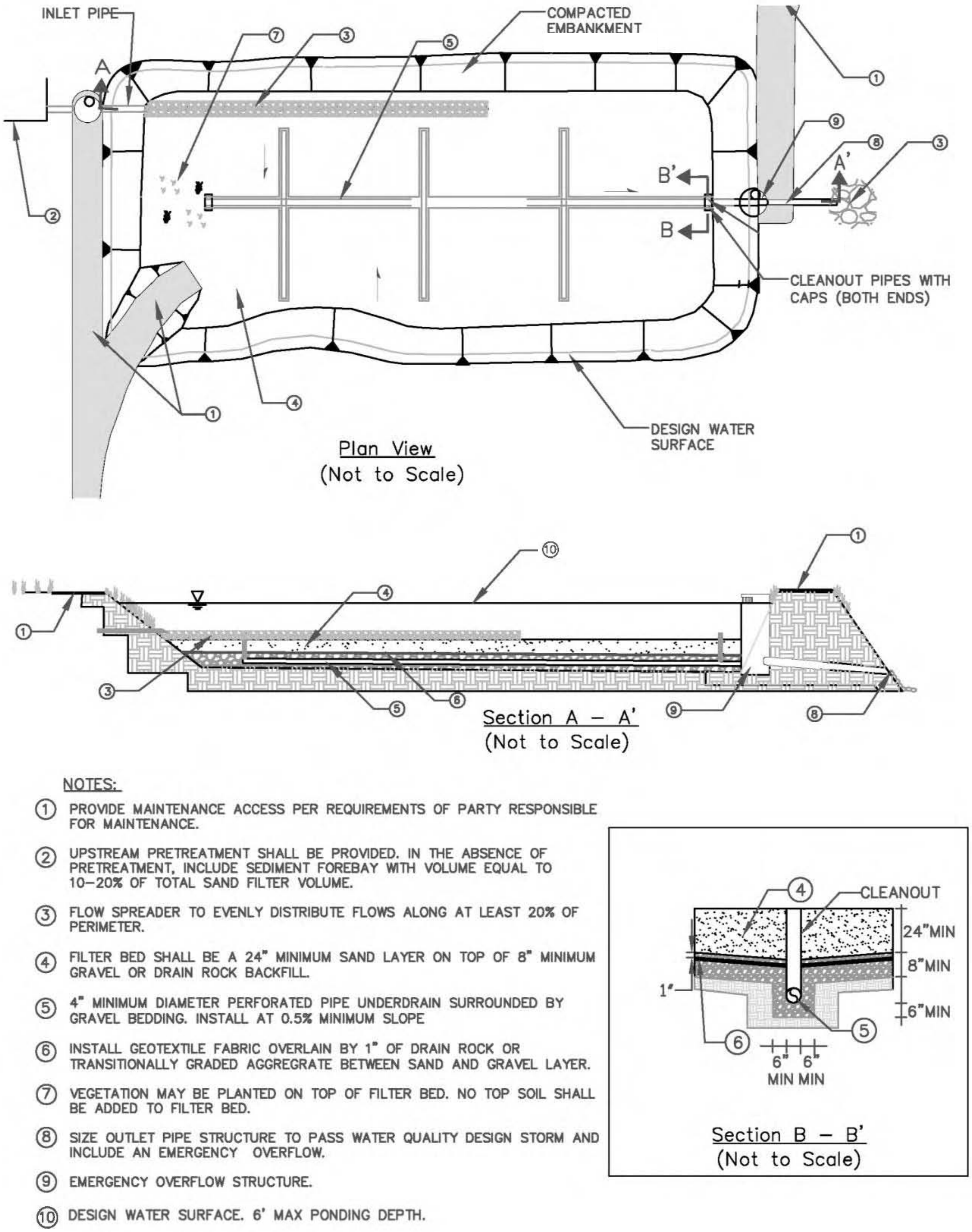
$n$ = Manning's roughness (use 0.011 for smooth pipe and 0.016 for corrugated pipe)

$s$ = pipe slope (recommended to be 0.005)

If more than one pipe is used, then this formula should be used to determine the sizing of the combination of pipes so that the sum of the flow rates of each pipe used is greater than or equal to $Q_{und}$.

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of media filters. Other designs are permissible if minimum design criteria are met.



## MAINTENANCE

Media bed filters are subject to clogging by fine sediment, oil and grease, and other debris (e.g., trash and organic matter such as leaves). Filters and pretreatment facilities shall be inspected every 6 months during the first year of operation. Inspections shall also occur immediately following a storm event to assess the filtration capacity of the filter. Once the filter is performing as designed, the frequency of inspection may be reduced to 2-3 times per year.

Cold weather may reduce the infiltration rates and the treatment effectiveness of media filters. Surface filters that retain large volumes of water due to clogging or high organic content are the most susceptible to freezing. Filters should be inspected before the first forecasted freeze to ensure clogging conditions in the fall do not evolve into frozen conditions in the winter.

Most of the maintenance shall be concentrated on the pretreatment practices (filter strips, vegetated swale or sedimentation forebay) upstream of the filter to ensure that sediment does not reach the filter. Regular inspection shall determine if the sediment removal structures require routine maintenance.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Remove trash, debris, and surficial sedimentation.<br>• Rake surface to break up silt crusts. |
| As needed (within 48 hours after every storm greater than 1 inch) | • Check for standing water.<br>• Check inlet structures for blockage.<br>• Remove any evidence of visual contamination from floatables such as oil and grease and dispose of properly. |
| As needed (infrequently) | • Clean and reset flow spreaders as needed to maintain even distribution of low flows.<br>• Remove minor sediment accumulation, debris, and obstructions near inlet and outlet structures as needed.<br>• Level the spreader and clean so that flows are spread evenly over the sand filter bed.<br>• Repair any tears in filter fabric. |
| Infrequently (when surface water no longer drains within 24 hours – typically about 3 to 5 years) | • Clean or back flush the drainage pipe, removing accumulated litter on surface or removing and renewing top 1-2” of filter media. If this does not cure problem then continue with steps below.<br>• Clean out underdrains if present to alleviate ponding. Replace filter bed media if ponding or loss of infiltrative capacity persists and re-vegetate as needed.<br>• Reset settled piping, add fill material to maintain original pipe flow line elevations<br>• Repair structural damage to flow control structures including inlet, outlet, and overflow structures. |

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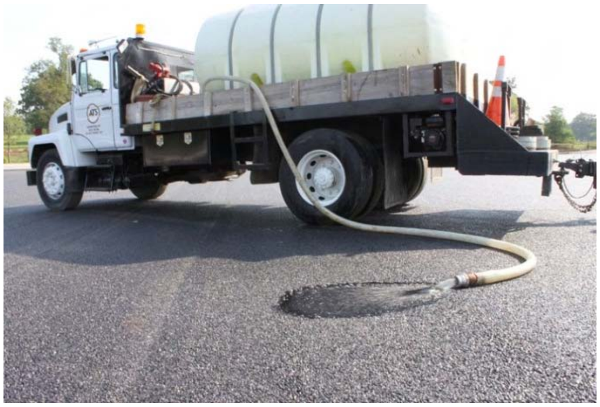
# PERMEABLE PAVEMENT

Structural Best Management Practice

Volume Control ☑

Quality Control ☑



*Porous pavement demonstration at Kentucky Horse Park (Photo from http://www.paiky.org/photogallery/show/24)*

| PERFORMANCE | | | |
|---|---|---|---|
| H | Sediment | M | Bacteria |
| M | Metals | L | Trash and debris |
| M | Oil and grease | L/M | Volume Reduction |
| M | Nutrients | M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

*Applications*

- Parking lots and driveways
- Low traffic roads
- Boat ramps
- Plazas and walking paths
- Outdoor athletic courts
- Golf cart paths

*Advantages*

✓ Allows runoff to infiltrate into subsoils; groundwater recharge
✓ Easily integrated into existing infrastructure
✓ Volume & peak flow reduction

*Limitations*

– Not ideal for high-traffic areas
– Not suitable for storm water hot spot sites
– Propensity to clog if not designed, constructed and maintained properly

PERMEABLE PAVEMENT

## DESCRIPTION

Permeable pavement is an alternative to conventional impervious asphalts and concretes. While conventional pavement types result in increased rates and volumes of surface runoff, permeable pavement, when properly constructed, allows some water to pass through into a subsurface gravel layer that acts both as a storage/infiltration area and a structural base layer. Where site conditions allow, the subsurface gravel layer (open-graded base/sub-base) is configured to allow water to infiltrate into the surrounding subsoil. If site conditions do not allow for infiltration, the water is detained in the gravel storage layer and then routed to a storm water conveyance system via an underdrain system. In either case, the initial infiltration of runoff through the surface layers increases the time of concentration, $T_c$, of the drainage area, provides some filtering of pollutants, and decreases the peak flows. When the water is allowed to infiltrate, it can also significantly decrease the volume of runoff leaving the site. Depending on the permeability or the volumetric moisture sensitivity (i.e., the plasticity) of the native soil infiltration rate, it may be necessary to install an impermeable liner below the base layer as well as an underdrain system. There are several styles of permeable pavement available, including those that are poured in place (i.e. porous concrete or porous asphalt), and modular paving systems (i.e. interlocking concrete, grass and gravel pavers).

PERMEABLE PAVEMENT

### *Poured in place permeable pavement*

Porous asphalt and porous concrete are poured where they will ultimately be used and are allowed to setup (cure) in place. Typically, the pore spaces in the pavement make up 15 - 35% of the total surface area. Porous asphalt and porous concrete are similar to each other in that the porosity is created by removing the small aggregate or fine particles from the conventional mix design, which leaves stable air pockets (gaps through the material) for water to drain through into the subsurface. Porous concrete is rougher than its conventional counterpart, and unlike oil-based asphalt, will not release harmful chemicals into the environment. These types of permeable pavements shall only be used in areas of slow and low traffic (e.g., parking lots, low traffic streets, pedestrian areas, etc.).

### *Modular paving systems*

There are several varieties of pavers that allow for infiltration, including (but not limited to) interlocking concrete pavers, grass pavers, gravel pavers, and permeable articulated concrete blocks/mats. Typically, the pore spaces in the pavement make up about 10% of the total surface area. Interlocking concrete pavers and permeable articulated concrete blocks/mats are not porous themselves; rather the mechanism that allows them to interlock creates voids and gaps between the pavers that are filled with a pervious material and can withstand heavy loads. Grass and gravel pavers are nearly identical to each other in structure (rigid grid of concrete or durable plastic) but differ in their load bearing support capacities. The grids are embedded in the soil to support the loads that are applied, thereby preventing compaction, reducing rutting and erosion. Grass pavers are generally filled with a mix of sand, gravel, and soil to support vegetation growth (e.g., grass, low-growing groundcovers, etc.), which provides habitat and pollutant removal, while reducing storm water runoff volumes and rates. Grass pavers are good for low-traffic areas, while gravel pavers are good for high-frequency, low speed traffic areas. Gravel pavers differ from grass pavers in that they are filled with gravel (often underlain with a geotextile fabric to prevent the migration of the gravel into the subbase) which support greater loads and higher traffic volumes.

## SITE SUITABILITY CONSIDERATIONS

Permeable pavements can be used in a number of places that conventional pavements are used, including low-traffic driveways and parking lots, sidewalks, walkways, plazas and paths, outdoor athletic courts, and golf cart trails. The following table summarizes general site suitability considerations for permeable pavements.

| SITE SUITABILITY CONSIDERATIONS FOR PERMEABLE PAVEMENTS | |
|---|---|
| Tributary Area | < 5 acres (217,800 $ft^2$) [1,2] |
| Typical BMP area as percentage of tributary area (%) | 25 – 100 percent |
| Site slope (%) | < 5 %[3] |
| Depth to bedrock or seasonally high groundwater table from bottom of aggregate layer | < 10 ft use underdrains |
| Hydrologic soil group | Any[4] |
| Distance from public/private wells | 200 ft |

1 – Tributary area is the area of the site draining to the BMP (including the area of the BMP itself). Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.
2 – Impervious surfaces draining to the BMP are limited to surfaces immediately adjacent to the permeable pavement, rooftop runoff, or other surfaces that do not contain significant sediment loads.
3 – If slope exceeds given limit or is within 200 feet from the top of a hazardous slope or landslide area, a geotechnical investigation is required. If a gravel base is used for storage of runoff: (1) slopes shall be restricted to 0.5% (steeper grades reduce storage capacity) and (2) underdrains shall be used if within 50 feet of a sensitive steep slope.
4 – Underdrains shall be provided for sites where measured soil infiltration rates are less than 2.0 in/hr.

The effectiveness of permeable pavement is related to the contributing land use, the size of the drainage area, the soil type, slope, drainage area imperviousness, and the pavement design and sizing. Permeable pavement can be combined with other storm water runoff BMPs to form a "treatment train" that can provide enhanced water quality treatment and volume reductions. Additional site suitability recommendations and potential limitations for permeable pavement are listed below.

- Pavement Type – The use of the area should be considered before selecting the permeable pavement type. For instance, pavers may not be a good option for locations where people may be walking in high heels, or where gravel from pavers could be displaced from vehicle tires onto nearby streets. Additionally, gravel-pavers shall not be placed on walkways that are required to be handicap accessible.
- Soils – Where possible, construct pavements in areas of uncompacted cut. Permeable pavement should be lined or avoided in areas where soils might be contaminated. Permeable pavement should not be located near steep or sensitive slopes without a geotechnical investigation to address the effects of the pavement system on these slopes. See Appendix D for more information regarding soil assessments and/or geotechnical investigations.
- Development density – Because permeable pavement can be placed in many locations where conventional pavement would normally be used, it is often a good option for denser developments.
- Adjacent Land Uses – Permeable pavement is not suitable for locations that are adjacent to industrial sites or "hotspot" locations where environmentally harmful releases may occur. Permeable pavement is also not recommended in areas which may produce a significant amount of sediment, or areas which may accumulate sand.

- Storm Drain – For permeable pavement systems with underdrains, site must have adequate relief between the pavement surface and the outlet of the underdrain system to permit vertical percolation through the gravel drainage layer and underdrain to the conveyance system.

PERMEABLE PAVEMENT

## DESIGN CRITERIA

Permeable pavement is designed to only treat the areas directly adjacent to the pavement surface. The main challenge associated with permeable pavement is sediment removal, which is critical to pavement performance. The following table summarizes the minimum design criteria for permeable pavement. Additional sizing criteria and design guidance is provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 for calculating $V_{wq}$ |
| Drawdown time for gravel drainage layer | hr | 48 (maximum) |
| Underdrain | -- | 6 inch minimum diameter; 0.5% minimum slope |
| Overflow Device | -- | Required |

### *Pretreatment*

Depending on how and where permeable pavement will be used, pretreatment of the runoff entering the pavement may be necessary. Permeable pavement should never accept run-on from areas that are not completely stabilized, and pretreatment is necessary when accepting runoff from any pervious surface. Without adequate pretreatment (typically a 5 foot vegetated filter strip buffer), clogging may significantly decrease the life of the permeable pavement. If sheet flow is conveyed to the permeable pavement over stabilized grassed areas, the site must be graded in such a way that minimizes erosive conditions. In general, the intended purpose of permeable pavement is to treat on-site areas only.

### *Geometry and Size*

Permeable pavement shall be sized to capture and treat the water quality design volume, $V_{wq}$.

Pavement design options include:

- Full or partial infiltration – A design for full infiltration uses an open graded base for maximum infiltration and storage of storm water. The water infiltrates directly into the base and through the soil. Pipes provide drainage in overflow conditions. Partial infiltration does not rely completely on infiltration through the soil to dispose of all of the captured runoff. Some of the water may infiltrate into the soil and the remainder drained by the underdrain system.

- No infiltration – No infiltration is desirable when the soil has low permeability and low strength, or there are other site limitations such as contamination or highly plastic soils. In such instances, an underdrain should be provided. By storing water for a time in the base and then slowly releasing it through pipes, the design behaves like an underground detention basin. In other cases, the soil of the sub-base may be compacted and stabilized to render improved support for vehicular loads. This practice reduces infiltration into the underlying soil to a negligible amount.

### *Pavement Layers*

Porous pavement systems generally consist of at least four different layers of material. The depth of each layer shall be determined by a licensed civil engineer based on analyses of hydrology, hydraulics, and structural requirements of the site.

- Top or wearing layer – Permeable pavement or pavers designed with voids to infiltrate or filter storm water to base layers. The thicknesses of these layers vary depending on design. Pavers shall have a minimum thickness of 3.125 inches.
- Bedding course layer - A layer of smaller sized aggregate (e.g. No. 8 or washed sand) just under the permeable pavement provides a level surface for installing the permeable pavement and also acts as a filter to trap particles and help prevent the reservoir layer from clogging. The bedding course layer is typically about 1.5 to 3 inches deep.
- Stone reservoir or aggregate layer – This layer, just below the bedding course layer, provides the bulk of water storage capacity for the permeable pavement system. This layer must be designed to function as a support layer as well as a reservoir layer. It is typically composed of washed, open-graded No. 57 aggregate without any fine sands. If no infiltration is allowed, an impermeable liner shall be placed under the subsurface gravel layer. The reservoir layer shall have zero slope (i.e. level).
- Transition layer(s) – Porous pavement design typically includes two or more transition layers. Generally a transition layer of either non-woven geotextile fabric or choking stone (typically No. 8 aggregate) s placed below the bedding course layer. This should be added at the discretion of the designer (e.g., if the bedding course layer is No. 8 aggregate and the stone reservoir layer uses No. 57 aggregate, then a transition layer is not needed here). In addition to this use, there will likely be a transition layer between the stone reservoir layer and the subsurface soil. These layers act as a filter to trap particles and help prevent underlying layers from clogging.



### Drainage

- Permeable pavement (including the aggregate and bedding course layers beneath) shall be designed to drain in less than 48 hours. The drawdown time is important because soils must be allowed to dry out periodically in order to restore hydraulic capacity. Adequate hydraulic capacity allows permeable pavement to receive flows from subsequent storms, maintain infiltration rates, maintain adequate subsoil oxygen levels for healthy soil biota, and to provide proper soil conditions for biodegradation and retention of pollutants.

### Underdrains

- If underdrains are required, then they must be made of perforated or slotted, polyvinyl chloride (PVC) pipe conforming to ASTM D 3034 or equivalent or corrugated high density polyethylene (HDPE) pipe conforming to AASHTO 252M or equivalent. Underdrains shall slope at a minimum of 0.5 percent, and smooth and rigid PVC pipes shall be used as underdrains with slopes of less than 2 percent.
- The perforations or slots shall be sized to prevent the migration of the drain rock into the pipes, and shall be spaced such that the pipe has a minimum of 1 square inch of opening per lineal foot of pipe.
- The underdrain pipe must have a 6-inch minimum diameter, so it can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe must be placed at the terminal ends of the underdrain. The cleanout risers shall be plugged with a lockable well cap. It is recommended to keep the cap locked in areas prone to vandalism.
- The underdrain shall be placed parallel to the pavement bottom. The underdrain shall be bedded with 6 inches of drain rock and backfilled with a minimum of 6 inches of drain rock around the top and sides of the

underdrain. The drain rock shall consist of clean, washed No. 57 stone, conforming to the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet, or an approved equal, that meets the gradation requirements listed in the table below.

| SIEVE SIZE | PERCENT PASSING |
|---|---|
| 1-$^1/_2$ inch | 100 |
| 1 inch | 95-100 |
| $^1/_2$ inch | 25-60 |
| US No. 4 | 0-10 |
| US No. 8 | 0-5 |

- The drain rock must be separated from the native soil layer below and to the sides with an approved non-woven geotextile fabric. The non-woven geotextile filter fabric should have a minimum flow rate of 50 gal/min/ft$^2$. Unless otherwise approved, the non-woven geotextile fabric shall conform to the Type II Fabric Geotextiles for Underdrains described in the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet. The minimum requirements for the non-woven geotextile filter fabric are provided below:

| GEOTEXTILE PROPERTY | VALUE | TEST METHOD |
|---|---|---|
| Grab Strength (lbs.) | 80 | ASTM D4632 |
| Sewn Seam Strength (lbs.) | 70 | ASTM D4632 |
| Puncture Strength (lbs.) | 25 | ASTM D4833 |
| Trapezoid Tear (lbs.) | 25 | ASTM D4533 |
| Apparent Opening Size US Std. Sieve | No. 50 | ASTM D4751 |
| Permeability (cm/s) | 0.010 | ASTM D4491 |
| UV Degration at 150 hrs. | 70% | ASTM D4355 |
| Flow Rate (gpm/ft$^2$) | 50 | ASTM D4491 |

- The underdrain pipe must drain freely to an acceptable discharge point.

### *Overflow*

An overflow mechanism is required. There are two overflow options for permeable pavement:

- Perimeter Control – Flows in excess of the design capacity of the permeable pavement system will require an overflow system connected to a downstream conveyance or other storm water runoff BMP. In addition, if the pavement becomes clogged and infiltration decreases to the point that there is ponding, the runoff will migrate off of the pavement via overland flow instead of infiltrating into the subsurface gravel layer. There are several options for handling overflow using perimeter controls such as: perimeter vegetated swale, perimeter bioretention, storm drain inlets and storm sewer, or rock filled trench that funnels flow around pavement and into the subsurface gravel layer.

- Overflow Pipe(s) – This overflow option involves connecting vertical pipes to the underdrain. The diameter, location, and quantity of pipe(s) vary with design and shall be determined by a licensed civil engineer. The overflow pipe(s) shall be located away from vehicular traffic. The top of the pipe(s) should be covered with a screen fastened over the overflow inlet. If desired, an observational and/or cleanout well may be incorporated into the pipe design.

## DESIGN PROCEDURE

The flow capacity of permeable pavement is usually limited by the infiltration rate of soils below it. The procedure below assumes that full or partial infiltration will be used. If the measured infiltration rate of soils is less than 2.0 in/hr (see Appendix D for example soil testing procedures), an underdrain is recommended to compensate for this. The underdrain may be placed near the top of the reservoir layer to enable partial infiltration/volume reduction to occur. In areas where geotechnical hazards or poor permeability preclude infiltration, the underdrain should be placed at the bottom of the reservoir layer and this layer may be decreased to one foot in thickness.

### *Step 1: Calculate the Water Quality Design Volume*

The water quality design volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3. Note that the tributary area should include the area of the permeable pavement plus any adjacent surfaces that drain to the pavement. The permeable pavement area should be assumed to be 100% impervious for the purposes of computing the water quality design volume. The ratio total tributary area (including the porous pavement) to the area of the porous pavement should not exceed 4:1 for permeable asphalt or concrete and 2:1 for permeable pavers. If there is no underdrain, larger drainage areas are permissible if the water quality design volume can be fully infiltrated and the tributary area yields low sediment loads.

### *Step 2: Design Infiltration Rate*

The design infiltration rate is based on the hydraulic conductivity of the native soil as determined using an in-situ percolation test measured at the elevation of the proposed bottom of the facility or at the depth of a limiting layer multiplied by a factor of safety of 0.25:

$$k_{native} = 0.25 \cdot k_{measured}$$

Where:

$k_{native}$ = the design infiltration rate for the native soils (in/hr)
$k_{measured}$ = the measured infiltration rate (in/hr)

If $k_{native}$ is less than 0.5 in/hr, then an underdrain is recommended.

### *Step 3: Determine the 48-hour Effective Depth*

Determine the effective depth of water that can be drawn down within 48 hours.

$$d_{48} = \left(\frac{48}{12}\right) \cdot k_{design}$$

Where:

$d_{48}$ = effective depth of water that can be drawn down in 48 hours (ft)
$k_{design}$ = design infiltration rate determined in Step 2 (in/hr).

### *Step 4: Determine the Aggregate Reservoir Depth*

The depth of water stored in the gravel reservoir (below the invert of the underdrain, if one is present, or below the pavement bedding course if no underdrain) should be equal or less than $d_{48}$. Determine the effective reservoir depth such that:

$$d_r \leq \frac{d_{48}}{\eta_r}$$

Where:

$d_{48}$ = effective depth of water that can be drawn down in 48 hours (ft)
$\eta_r$ = porosity of aggregate reservoir fill (unitless) [use 0.32 unless aggregate-specific data available]
$d_r$ = depth of gravel drainage layer below the invert of the underdrain, if present, or below the bedding course if no underdrain (ft)

PERMEABLE PAVEMENT

### *Step 5: Calculate the Required Infiltrating Area*

The required infiltrating area for complete infiltration of the water quality design volume can be calculated using the following equation:

$$A_{inf} \geq V_{wq} / (n_r \times d_r)$$

Where:
$A_{inf}$ = required infiltration area ($ft^2$)
$V_{wq}$ = water quality design volume ($ft^3$)
$n_r$ = porosity of aggregate reservoir fill (unitless)
$d_r$ = depth of gravel drainage layer below the base of the underdrain, if present, or below the bedding course if no underdrain (ft)

If $A_{inf}$ is less than the planned permeable pavement area, the drainage area may be increased (repeat Steps 1 and 5 to do this). If $A_{inf}$ is greater than the planned permeable pavement area, then the drainage area must be decreased. As a rule of thumb, the ratio of total tributary area (including the porous pavement) to the area of the porous pavement should not exceed 4:1 for porous asphalt or concrete and 2:1 for porous pavers. If there is no underdrain, larger drainage areas are permissible if the water quality design volume can be fully infiltrated and the tributary area yields low sediment loads. If there is an underdrain and the computed $d_r$ is less than 6 inches, the tributary area ratio does not need to be reduced below the maximum ratios listed above.

### *Step 6: Flow Capacity of Underdrain*

Underdrains must be designed so they drain water from the rock layer quickly enough that the pavement above does not flood. The design flow capacity of the underdrain pipe can be computed as:

$$Q_{und} = f_s \frac{k_{media} \cdot A}{(12)(3600)}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$f_s$ = factor of safety [use 3]

$k_{media}$ = design infiltration rate (in/hr) [use 2 in/hr]

$A$ = area of permeable pavement or infiltration area ($ft^2$)

### *Step 7: Number of Underdrain Pipes*

The diameter of a single pipe to convey the underdrain flow can be computed as:

$$D_s = 16 \cdot \left( \frac{Q_{und} \cdot n}{s^{0.5}} \right)^{3/8}$$

Where:

$D_s$ = single pipe diameter (in)

$Q_{und}$ = required flow capacity of underdrain (cfs)

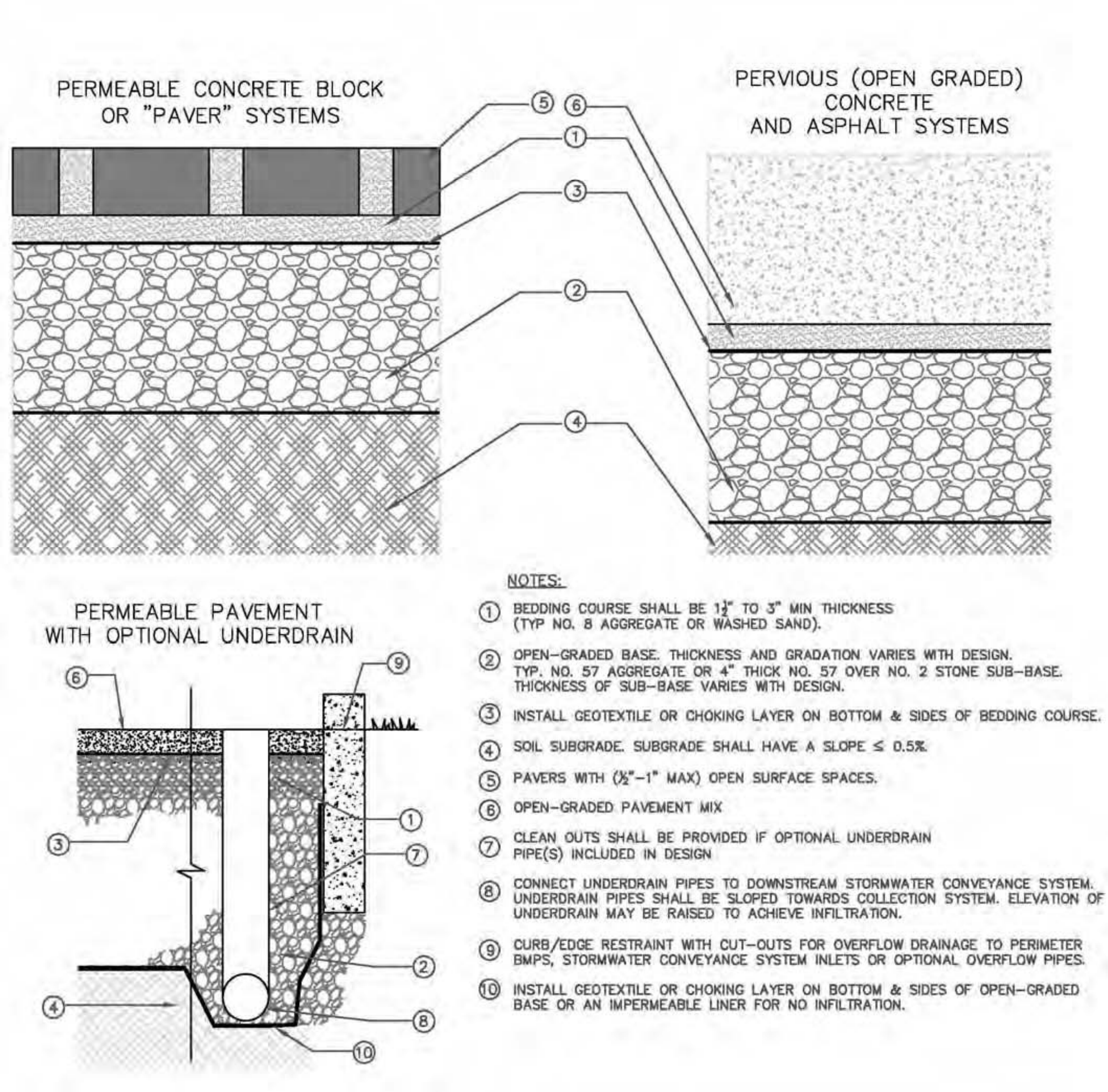
$n$ = Manning's roughness (use 0.011 for smooth pipe and 0.016 for corrugated pipe)

$s$ = pipe slope (recommended to be 0.005) (ft/ft)

If more than one pipe is used, then this formula should be used to determine the sizing of the combination of pipes so that the sum of the flow rates of each pipe used is greater than or equal to $Q_{und}$ .

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of porous pavement. Other designs are permissible if minimum design criteria are met.




Note that portions of these design schematics were modified from the City of Portland Stormwater Management Manual.

## MAINTENANCE

Maintenance crews should be reminded not to use sand in winter deicing operations and should be educated in proper maintenance procedures.  Porous pavement should never be seal-coated.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Remove any trash and debris accumulated on pavement surface.<br>• Manage or stabilize vegetated areas adjacent to pavement such that no bare soil is exposed. |
| As needed (within 48 hours after every storm greater than 1 inch) | • Inspect pavement for surface ponding.<br>• Inspect overflow structure(s) for clogs.<br>• Remove/mitigate visual contaminants or pollutants.<br>• Inspect tributary areas for signs of erosion or instability and stabilize as needed.<br>• For winter conditions: salt and/or sand shall not be used; avoid plowing for snow removal |
| As needed (infrequently) | • Repair cracks, depressions, or crumbling visible on pavement surface.<br>• Mitigate subsurface clogs (i.e. those that are not remedied by addressing surface clogging or underdrain cleanout) by excavating gravel drainage layer and cleaning up clog.  Replace surface porous pavement and underlying layers |
| Semi-annually | • Remove visible sediment accumulation: Vacuum 2-4 times/yr<br>• Fill in interstitial gaps between pavers with gravel/sand fill.<br>• Remove all vegetative growth in permeable pavements except for within grass pavers. |

PERMEABLE PAVEMENT

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# PLANTER BOX

Structural Best Management Practice



Geosyntec Consultants

| PERFORMANCE | | | | | |
|---|---|---|---|---|---|
| H | Sediment | | H | Bacteria |
| H | Metals | | H | Trash and debris |
| M | Oil and grease | | L | Volume Reduction |
| L/M | Nutrients | | M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☐

Quality Control ☑

*Applications*

- Commercial and institutional
- Areas adjacent to buildings and walkways
- School entrance and walkways
- Parking lots

*Advantages*

✓ Combines storm water treatment with runoff conveyance
✓ Peak flow reduction
✓ Easily incorporated into site landscaping

*Limitations*

– May require additional support on steep slopes
– Must be constructed with underdrain to convey excess water to storm water conveyance system
– Not suitable for large drainage areas

## DESCRIPTION

Planter boxes are structurally contained bioretention facilities designed to capture and temporarily store storm water runoff. These facilities function as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological and chemical treatment processes as runoff percolates through the planter boxes. Planter boxes consist of a gravel underdrain system, boundary layer of geotextile or sand/choking stone, planting soil media, and vegetation. The planter box structure itself may be comprised of a variety of materials (usually chosen to be the same materials as adjacent building or sidewalk).

Planter boxes may be placed adjacent to buildings or other structures and beneath downspouts as long as the boxes are properly lined on the building side and the overflow outlet discharges away from the building to ensure water does not percolate into footings or foundations. They can also be placed further away from buildings by conveying roof runoff in shallow engineered open conveyances, shallow pipes, or other innovative drainage structures.

## SITE SUITABILITY CONSIDERATIONS

Planter boxes are uniquely suited for denser urban areas, including redevelopment projects. In addition, planter boxes are suitable for sites where infiltration practices are limited, impractical or discouraged due to poorly draining soils, setback limitations, or other constraints. Planter boxes are often designed to capture runoff from rooftop downspouts of commercial, industrial, and residential structures and offer peak discharge rate reduction and some volume reduction of roof drainage via evapotranspiration (and possibly infiltration if allowed).

| SITE SUITABILITY CONSIDERATIONS FORPLANTER BOXES | |
|---|---|
| Tributary Area | < 0.35 acres; 15,000 ft[2] |
| Typical BMP area as percentage of tributary area (%) | < 5 percent |
| Site slope (%) | < 6 percent[2] |
| Depth to seasonally high groundwater table below planter box bottom | > 2 ft |
| Hydrologic soil group | Any [3] |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.
2 – If longitudinal slope is greater than 6%, then terracing of planter boxes is recommended.
3 – Typical systems are assumed to be lined with an underdrain. If infiltration is incorporated, the site soils must have a measured infiltration rate of 2 in/hr at the location where infiltration is planned. Site must have adequate relief between land surface and the storm water conveyance system to permit vertical percolation through the gravel drainage layer (open-graded base/sub-base) and underdrain to the storm water conveyance system.

Planter boxes have a wide range of applicability in terms of site suitability, but are best used in tributary areas that are smaller and highly urban. Planter boxes can also be used in “treatment train” applications. For example, if a planter box is placed upgradient of a cistern, the rate and volume of water flowing to the cistern can be reduced and the water quality enhanced. As another example, a planter box could be placed downstream of a downspout that drains the green roof. Other site suitability issues are included below:

- Placement – Placement of planter boxes should take into account where water is entering and exiting the system. Proper positioning of overflow devices is important to ensure that excess water does not accumulate around structure footing and foundations. Additionally, there should be adequate relief between the planter box and the storm water conveyance system to ensure suitable drainage. When properly planned, planter boxes can be located directly adjacent to structures and buildings.
- Development density – Planter boxes are well-suited for dense developments because they are not subject to setback requirements and other constraints that may preclude the use of other BMPs near buildings, sidewalks, and other structures (as long as they are fully lined).
- Adjacent Land Uses – Planter boxes can be used for commercial, multi-family, and institutional land uses and often can be well-incorporated into the landscaping. Planter boxes are not recommended for industrial areas or areas which may have high sediment or pollutant loading. If placed next to sidewalks, a curb must be installed or the box must be raised to protect pedestrians.
- Shade –For highly shaded areas, shade tolerant plants and grasses shall be used.

## DESIGN CRITERIA

The following table summarizes the minimum design criteria for planter boxes. Additional sizing criteria and design guidance is provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 for calculating $V_{wq}$ |
| Maximum ponding depth | in | 8; 6 preferred |
| Planting soil depth | ft | 2; 3 preferred (required with trees) |
| Stabilized mulch depth | in | 2 to 3 |
| Planting media composition | - | See Appendix B |
| Underdrain | - | 4" minimum diameter; 0.5% minimum slope |
| Vegetation type | - | Varies (see vegetation section below and Appendix C) |

PLANTER BOX

### *Geometry and Size*

- Planter boxes areas shall be sized to capture and treat the water quality design volume, $V_{wq}$, with an 8-inch maximum ponding depth. When space allows, the preferred maximum ponding depth is 6 inches.
- Planting soil depth shall be a minimum of 2 feet, although 3 feet is preferred in most cases and required if trees have been planted in the planter box. This planting soil depth shall provide a beneficial root zone for the chosen plant palette and adequate water storage for the water quality design volume.
- At the end of rainfall, planter boxes should drain ponded water in less than 12 hours and all free water contained in the planting media should drain in less than 24 hours. This drawdown time is important as soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, prevent long periods of saturation for plant health, maintain adequate soil oxygen levels for healthy soil biota and vegetation, reduce potential for vector breeding, and to provide proper soil conditions for biodegradation and retention of pollutants.
- Any planter box shape configuration is possible as long as other design criteria are met.
- To increase the opportunity for storm water retention and filtration, the distance between the downspouts and the overflow outlet should be maximized.

### *Planter Box Structural Materials*

- Planter boxes shall be constructed out of stone, concrete, brick, recycled plastic, or other permanent materials. Pressure-treated wood or other materials that may leach pollutants (e.g., arsenic, copper, zinc, etc.) shall not be allowed.
- The structure should be adequately sealed or a waterproof membrane installed to ensure water only exits the structure via the underdrain. Geomembrane liners shall have a minimum thickness of 30 mils. Equivalent waterproofing measures may be used. In some cases, unlined planter boxes may be used if the underlying soils are conducive to infiltration (>2 in/hr measured infiltration rate) and the structural integrity of the adjacent buildings or roads will not be impacted.

### *Inflows and Energy Dissipation*

- Piped entrances, such as roof downspouts, shall include rock, splash blocks, or other appropriate measures at the entrance to dissipate energy and disperse flows.

- Woody plants (e.g., trees, shrubs, etc.) can restrict or concentrate flows and can be damaged by erosion around the root ball and shall not be placed directly in the entrance flow path.

### *Underdrains*

- Underdrains must be made of perforated or slotted, polyvinyl chloride (PVC) pipe conforming to ASTM D 3034 or equivalent or corrugated high density polyethylene (HDPE) pipe conforming to AASHTO 252M or equivalent. Underdrains shall slope at a minimum of 0.5 percent, and smooth and rigid PVC pipes shall be used as underdrains with slopes of less than 2 percent.
- The perforations or slots shall be sized to prevent the migration of the drain rock into the pipes, and shall be spaced such that the pipe has a minimum of 1 square inch of opening per lineal foot of pipe.
- The underdrain pipe must have a 4-inch minimum diameter, which is smaller than other water quality BMPs due to the small drainage areas that are typically tributary to planter boxes. Clean-out risers with diameters equal to the underdrain pipe must be placed at the terminal ends of the underdrain. The cleanout risers shall be plugged with a lockable well cap. It is recommended to keep the cap locked in areas prone to vandalism.
- The underdrain shall be placed along the long axis of the planter box. The underdrain shall be bedded with 6 inches of drain rock and backfilled with a minimum of 6 inches of drain rock around the top and sides of the underdrain. The drain rock shall consist of clean, washed No. 57 stone, conforming to the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet, or an approved equal, that meets the gradation requirements listed in the table below.

| SIEVE SIZE | PERCENT PASSING |
|---|---|
| 1-$^1/_2$ inch | 100 |
| 1 inch | 95-100 |
| $^1/_2$ inch | 25-60 |
| US No. 4 | 0-10 |
| US No. 8 | 0-5 |

- The drain rock must be separated from the native soil layer below and to the sides with an approved non-woven geotextile fabric. The drain rock shall be separated from the planting media above with an approved non-woven geotextile fabric or with an appropriately graded granular filter. The graded granular filter should consist of 2 to 4 inches of washed sand underlain with a minimum 2 inches of choking stone (washed No .8 or No. 89 pea gravel). The non-woven geotextile filter fabric should not impede the infiltration rate of the planting media and should have a minimum flow rate of 50 gal/min/ft$^2$. Unless otherwise approved, the non-woven geotextile fabric shall conform to the Type II Fabric Geotextiles for Underdrains described in the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet. The minimum requirements for the non-woven geotextile filter fabric are listed in the table below.

| GEOTEXTILE PROPERTY | VALUE | TEST METHOD |
|---|---|---|
| Grab Strength (lbs.) | 80 | ASTM D4632 |
| Sewn Seam Strength (lbs.) | 70 | ASTM D4632 |
| Puncture Strength (lbs.) | 25 | ASTM D4833 |
| Trapezoid Tear (lbs.) | 25 | ASTM D4533 |
| Apparent Opening Size<br>US Std. Sieve | No. 50 | ASTM D4751 |
| Permeability (cm/s) | 0.010 | ASTM D4491 |
| UV Degration at 150 hrs. | 70% | ASTM D4355 |
| Flow Rate (gpm/ft$^2$) | 50 | ASTM D4491 |

- The underdrain pipe must drain freely to an acceptable discharge point.

### *Overflow*

- An overflow device is required to be set at least 2" below the top of the planter box and no more than 12 inches above the soil surface (6 inches preferred for aesthetics). The most common option is a vertical riser, described below.
- A vertical PVC pipe (SDR 35) shall be connected to the underdrain.
- The overflow riser(s) shall be 4 inches or greater in diameter, so it can be cleaned without damage to the pipe. The vertical pipe will provide access to cleaning the underdrains.

### *Planting/ Storage Media*

- The planting matrix of a planter box must provide stability and adequate support for proposed vegetation. It must be highly permeable and provide sufficient organic content and topped with a mulch layer 2-4 inches thick. The mulch layer should be shredded hardwood mulch or chips, aged a minimum of 12 months.
- The planter box should contain of a minimum of 2 feet (3 feet is preferred) of bioretention soil mix above the underdrain. See Appendix B for guidance on bioretention soil mixes.

### *Vegetation*

- Prior to installation, a licensed landscape architect shall be consulted to ensure that proposed plants are tolerant of drought, ponding fluctuations, saturated soil conditions, and additional light intensity that may result from building face reflection.
- Shade trees shall have a single main trunk. Trunks shall be free of branches below the following heights:

| CALIPER (IN) | HEIGHT (FT) |
|---|---|
| 1-1/2 to 2-1/2 | 5 |
| 3 | 6 |

- A variety of tree, shrub, and herbaceous groundcover species should be incorporated to protect against facility failure due to disease and insect infestations of a single species. Plant species and plant placement shall account for rooting depths to assure that the underdrain system is not damaged. Slotted or perforated underdrain pipe should be more than 5 feet from tree locations (if space allows).
- Prohibited non-native plant species shall not be used. Refer to the Boone County Zoning Regulations (Landscaping section) for a list of prohibited plant species. Further information on invasive plant species in

Kentucky can be found at the Early Detection & Distribution Mapping System (http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky).

- Plants must be healthy and vigorous. Within 2 years, a survival rate of 75 percent (no replacements) must be achieved. If the survival rate falls below this threshold, additional plants sufficient to meet the 75 percent survival rate must be installed.
- Only slow-release fertilizers shall be used to limit the potential for excessive nutrient discharge.
- Annual replacement of 1-2 inches of mulch (may be the same as compost used for planting mix) is required to sustain nutrient levels, suppress weeds, and maintain infiltrative capacity.

# DESIGN PROCEDURE

Because the bioretention soil media used in planter boxes (see Appendix B) has a high hydraulic conductivity, the 8" maximum ponding depth will drain in a relatively short period of time (less than eight hours when properly functioning), making the need for drawdown calculations unnecessary. The volume that can be treated in a planter box is therefore a function of the ponding depth and the surface area of the box.

PLANTER BOX

### Step 1: Water Quality Design Volume

The water quality design volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

### Step 2: Planter Box Surface Area

The required filter area can be calculated using the following equation:

$$A = \frac{12V_{wq}}{d_{pond} + (\eta)(d_{media})}$$

Where:

$A$ = required area of planter box ($ft^2$)

$V_{wq}$ = water quality design volume ($ft^3$)

$d_{pond}$ = design depth of ponding above planter box soil (in); should be 8 inches or less.

$\eta$ = drainable porosity of the media (unitless); use 0.25

$d_{media}$ = design depth of planter box soil (in); should be 24 inches or greater

### Step 3: Flow Capacity of Underdrain

Underdrains must be designed so they drain water from the rock layer substantially faster than water enters from the media layer above. The design flow capacity of the underdrain pipe can be computed as:

$$Q_{und} = f_s \frac{k_{design}(A)}{12(3600)}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$f_s$ = factor of safety (use 5)

$k_{media}$ = design infiltration rate of bioretention soil mix (use 2 in/hr)

$A$ = required area of planter box ($ft^2$)

### Step 4: Number and size of Underdrain Pipes

The diameter of a single pipe to convey the underdrain flow can be computed as:

$$D_s = 16\left(\frac{(Q_{und})(n)}{s^{0.5}}\right)^{3/8}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$D_s$ = single pipe diameter (in); minimum = 4”

$n$ = Manning’s roughness (use 0.011 for smooth pipe and .016 for corrugated pipe)

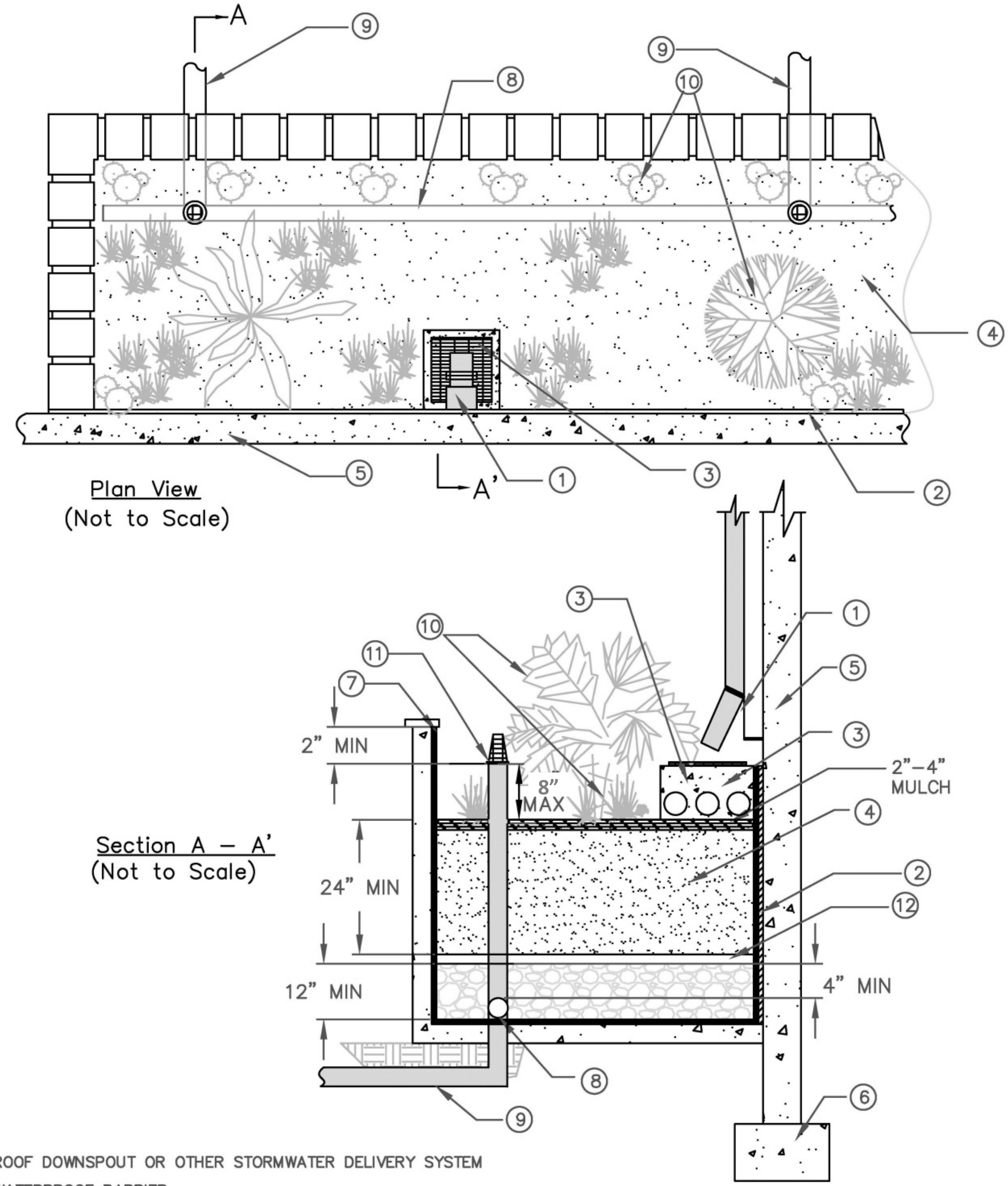
$s$ = pipe slope (recommended to be 0.005)



If more than one pipe is used, then this formula should be used to determine the sizing of the combination of pipes so that the sum of the flow rates of each pipe used is greater than or equal to $Q_{und}$.

## DESIGN SCHEMATICS

The following schematic should be used as further guidance for design of planter boxes. Other designs are permissible if minimum design criteria are met.




NOTES:

1. ROOF DOWNSPOUT OR OTHER STORMWATER DELIVERY SYSTEM
2. WATERPROOF BARRIER
3. INSTALL SHALLOW ENERGY DISSIPATOR BASIN OR EQUIVALENT TO DISPERSE FLOW AT SOIL SURFACE
4. BIORETENTION SOIL MIX
5. BUILDING
6. FOUNDATION. INSTALL FOUNDATION DRAINS AS NEEDED
7. OPTIONAL 30–MIL LINER OR EQUIVALENT
8. PERFORATED PIPE SHALL RUN ENTIRE LENGTH OF PLANTER
9. CONNECTION TO DOWNSTREAM CONVEYANCE SYSTEM
10. PLANTS
11. SET INVERT OF OVERFLOW 2" MIN BELOW THE TOP OF THE PLANTER
12. SAND AND CHOKING GRAVEL LAYER OR GEOTEXTILE

## MAINTENANCE

Planter boxes require periodic plant and planting media maintenance for aesthetics and continued performance. A majority of the maintenance activities required are typical of landscaped areas.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Remove any visual contaminants and pollutants.<br>• Maintain health of plants and remove any plants that interfere with the function of the planter box.<br>• Remove any trash and debris that has accumulated in the planter box. |
| As needed (within 48 hours after every storm greater than 1 inch) | • Inspect inlet and overflow and remove any material that blocks or clogs these areas.<br>• Inspect vegetated area for erosion and repair damaged areas.<br>• Inspect planter box for standing water that does not drain freely (within 24 hours after a storm event). Clean out underdrain, to alleviate ponding. |
| As needed (infrequently) | • Remove accumulation of fine sediment, dead leaves, etc. and replace with fresh mulch to restore surface permeability.<br>• Repair structural damage to flow control structure, including inlet, outlet, overflow, etc. Clean out underdrain as needed to alleviate standing water issues<br>• Re-grade and re-vegetate to repair damage from major erosion if needed.<br>• Replace media (if ponding or loss of infiltrative capacity persists) and re-vegetate. |

## ADDITIONAL SOURCES OF INFORMATION

AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001.

Boone County Planning Commision. Boone County Subdivision Regulations. 2010. http://www.boonecountyky.org/pc/2010SubdivisionRegs/2010SubRegs.pdf.

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City of Portland Bureau of Environmental Services. George Middle School, http://www.portlandonline.com/bes/index.cfm?a=196784&c=44953.

City of Portland, Oregon. Stormwater Management Manual. 2008. http://www.portlandonline.com/bes/index.cfm?c=47953&

Nevue Ngan Associated et al. Stormwater Management Handbook – Implementing Green Infrastructure in Northern Kentucky Communities. http://www.sd1.org/Resources.aspx?cid=3

Nashville, Tennessee. Stormwater Management Manual, Volume 4. 2009. http://www.nashville.gov/stormwater/regs/SwMgt_ManualVol04_2009.asp

Prince Georges County Bioretention Manual, 2009. http://www.princegeorgescountymd.gov/Government/AgencyIndex/DER/ESG/Bioretention/bioretention.asp

Sanitation District No. 1. Northern Kentucky Regional Storm Water Management Program: Rules and Regulations. 2011. Available at http://www.sd1.org/Resources.aspx?cid=9

Strecker, Eric and Klaus Rathfelder. Memo to Kentucky Sanitation District No. 1, Fort Wright, KY, 17 Nov. 2008.

U.S. EPA. National Menu of Stormwater Best Management Practices. http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm

Virginia Department of Conservation and Recreation. Virginia DCR Stormwater Design Specification No. 9: Bioretention. 2010. (refer to Appendix 9-A: Urban Bioretention).

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# RETENTION BASIN/WET POND

Structural Best Management Practice



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| PERFORMANCE | | | |
|---|---|---|---|
| H | Sediment | M | Bacteria |
| H | Metals | H | Trash and debris |
| M | Oil and grease | L | Volume Reduction |
| M | Nutrients | M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☐

Quality Control ☑

*Applications*

- Regional detention & treatment
- Commercial, residential
- Parks, open spaces, and golf courses

*Advantages*

- ✓ May be combined with flood control
- ✓ Suspended solids and particulate-bound pollutant removal
- ✓ May address dissolved constituents and nutrients
- ✓ Aesthetically pleasing
- ✓ Can provide treatment for large tributary areas

*Limitations*

- – Supplemental water may be required if water level is to be maintained
- – Large footprint area
- – Mosquito control may be required

## DESCRIPTION

Retention basins are constructed, naturalistic ponds with a permanent or seasonal pool of water (also called “wet pool” or “dead storage”). Aquascape facilities, such as artificial lakes, are a special form of wet pool facility that can incorporate innovative design elements to allow them to function as a storm water treatment facility in addition to an aesthetic water feature. Retention basins require base flows to exceed or match losses through evaporation and/or infiltration and they must be designed with the outlet positioned and/or operated in such a way as to maintain a permanent pool. Retention basins can be designed to provide extended detention of incoming flows using the volume above the permanent pool surface.

The benefits of retention basins are similar to those of dry extended detention (ED) basins and include peak flow attenuation (with ED), varying amounts of volume reduction, and pollutant removal. The main pollutant removal mechanism in retention basins is sedimentation; other pollutant reduction processes occurring in retention basins include adsorption and biochemical processes such as microbially-mediated transformations (e.g., biodegradation and precipitation) and plant uptake and storage. The permanent pool of water in the retention basins improves treatment of fine particulates and associated pollutants and provides treatment of dry weather flows. Permanent pools also allow retention basins to be designed as aesthetically pleasing water features with additional recreational, wildlife habitat, and educational benefits. A well-designed retention basin provides improved water quality treatment by increasing the average hydraulic residence time of storm water in the facility.

Retention basins work best under plug flow conditions where the water already present in the permanent pool is displaced by incoming flows with minimal mixing and no short circuiting. Plug flow describes the hypothetical condition of storm water moving through the basin in such a way that older "slugs" of water (meaning water that's been in the basin for longer) are displaced by incoming slugs of water with little or no mixing in the direction of flow. Short circuiting occurs when quiescent areas or "dead zones" develop in the basin where pockets of water remain stagnant, causing incoming storm water to bypass these zones). Longer residence times (and thus better water quality) are achieved when the permanent wet pool volume is greater than or equal to the water quality design volume.

## SITE SUITABILITY CONSIDERATIONS

Retention basins are volume-based BMPs intended to provide water quality treatment and, when extended detention is provided, attenuate peak runoff discharge rates. Retention basins can be applied to any location where sufficient space is available to treat larger tributary areas. Retention basins ideally have consistent base flows (at least seasonally) and they must be designed with the outlet positioned and/or operated in such a way as to maintain a permanent pool of water. In highly permeable soils, the basin may need to be lined in order for base flows to match or exceed infiltration losses. A liner may also be needed in wellhead protection areas to prevent surface water / groundwater interactions.

| SITE SUITABILITY CONSIDERATIONS FOR RETENTION BASINS | |
|---|---|
| Tributary Area[1] | > 10 acres (435,600 $ft^2$) |
| Typical BMP area as percentage of tributary area (%) | 2-5 percent |
| Proximity to steep sensitive slopes | Basins placed on slopes greater than 15 percent or within 200 feet from a hazardous slope or landslide area require a geotechnical investigation |
| Depth to seasonally high groundwater table | Not Applicable; A liner may be required if basin is located in a wellhead protection area. |
| Hydrologic soil group | Any[2] |
| Distance from public/private wells | 200 ft |
| Depth to bedrock | >2 ft |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.
2 – "A" Soils may require a pond liner. "B" soils may require infiltration testing to ensure base flows match or exceed losses. Check with Boone County regulations and apply the more stringent requirement.



The effectiveness of a retention pond is directly related to the contributing land use, the size of the drainage area, the soil type, slope, drainage area imperviousness, proposed vegetation, and the pond dimensions. Natural low points in the topography are well-suited for retention pond locations. Additional site suitability recommendations and potential limitations for retention ponds are listed below.

- Placement – Retention basins typically are used for treating areas larger than 10 acres and less than 10 square miles. They are especially appropriate for regional water quality treatment and flow control. Off-line retention basins must not interfere with flood control functions of existing conveyance and detention structures. If retention basins are located in areas with site slopes greater than 15% or within 200 feet of a hazardous steep slope or mapped landslide area, a geotechnical investigation and report must be provided to ensure that the basin does not compromise the stability of the site slope or surrounding slopes. Retention basins require a regular source of base flow if water levels are to be maintained. If base flow is insufficient during summer months, supplemental water may be necessary to maintain water levels.
- Soils – Liners should be considered in retention basin implementations in areas with high permeability soils. A water balance assessment should be used to confirm whether a liner is required to keep water in the wetlands (see Design Procedure section below). The liner will increase the chances of maintaining a permanent pool in the basin and protect groundwater quality. Conduct one test pit or boring per every 2 acres of permanent pool footprint, with a minimum of two per pond. Include information on the soil texture, color, structure, moisture and groundwater indicators, and bedrock type and condition, and identify all by

elevation. Liners can be either synthetic materials or imported lower permeability soils (i.e., clays). Wet ponds are not recommended in or near karst terrain.

- Development density – The retrofit of retention ponds into highly developed areas is typically not feasible due to the large space requirements needed for effective treatment and storage. New developments can often incorporate retention ponds as aquascape features into residential and office park developments.
- Adjacent Land Uses – Refer to local zoning ordinances for setback requirements for buildings and other structures from the high water level of any retention basin. Retention basins effectively mitigate flows and improve water quality for residential, commercial, and industrial areas. Many industrial facilities include retention ponds as part of a chemical spill containment plan and would dictate the need for a liner.

## DESIGN CRITERIA

The main challenge associated with retention basins is maintaining desired water levels. Additional design parameters can be found in the following table.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Flood control design discharge rate, $Q_{fc}$ | cfs | See SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. |
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 for calculating $Q_{wq}$ |
| Drawdown time for extended detention (over permanent pool) | hr | 24-36 |
| Depth without sediment storage | ft | 3 - 5 (forebay)<br>5-7 (main basin) |
| Depth with sediment storage | ft | 5 – 7 (forebay)<br>6 – 8 (main basin) |
| Freeboard (minimum) above max water level | in | 12 (off-line); 12 min (24 preferred) (on-line) |
| Flow path length to width ratio | L:W | 1.5:1 (min.) 3:1 (preferred) |
| Side slope (maximum) | H:V | Interior: 4:1 (H:V)<br>Exterior: 3:1 (H:V) (4:1 if mowed) |
| Longitudinal slope in the direction of flow | % | 1 (forebay) and 0-2 (main basin) |
| Vegetation Type | -- | Varies. See vegetation section below |
| Vegetation Height | -- | Varies. See vegetation section below |
| Buffer zone (minimum) | ft | Conform with local zoning ordinances/regulations |



### *Sizing for Meeting the Storm Water Runoff Requirements*

Retention basins can be sized to meet all or part of the water quality design volume as outlined in Chapter 3 and peak runoff discharge rate requirements as outlined in SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

- The retention basin can be designed with extended detention (above the permanent pool) to provide sufficient storage for meeting all or part of the peak runoff discharge requirement for the 2, 10, 25, 50, and 100-year design storms. For online basins that also provide flood control, the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations must also be met.
- The retention basin can be designed with or without extended detention (above the permanent pool) to treat all or part of the water quality treatment volume. If extended detention is provided, the drawdown time for the surcharge volume above the permanent pool should be 24 to 36 hours.

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### *Geometry and Size*

- If there is no extended detention provided, retention basins should be sized to provide a minimum wet pool volume equal to the water quality design volume plus an additional 2 feet (minimum) of depth for sediment accumulation in the forebay and 1 foot (minimum) in the main basin. If extended detention is provided above the permanent pool and the basin is designed for water quality treatment only, then the permanent pool volume should be a minimum of 10 percent of the water quality design volume and the surcharge volume (above the permanent pool) should make up the remaining 90 percent. The extended detention portion of the retention basin above the permanent pool, if provided, functions like a dry extended detention basin (see Chapter for dry ED basin sizing guidelines).

- Retention basins with wet pool volumes less than or equal to 4,000 cubic feet may be single-celled (i.e., no baffle or berm is required).
- Additional sediment storage should be provided in the forebay. The sediment storage should have a minimum depth of 2 feet. This volume should not be included as part of the required water quality volume.
- The minimum depth of the forebay should be 3 - 5 feet, exclusive of sediment storage requirements.
- The maximum depth of the main basin should not exceed 8 feet.
- At least 25% of the basin area should be deeper than 3 feet to prevent the growth of emergent vegetation across the entire basin.
- A retention basin should have a surface area of not less than 0.3 acres for each acre-foot of permanent pool volume. In addition, extra area needed to provide a design that meets all other provisions of this section should be provided. Additional surface area in excess of the minimum may be provided. There is no maximum surface area provided that all provisions of this section are met.
- Inlets and outlets should be placed to maximize the flow path through the facility. The flow path length-to-width ratio should be a minimum of 1.5:1, but a flow path length-to-width ratio of 3:1 or greater is preferred. The flow path length is defined as the distance from the inlet to the outlet, as measured at mid-depth of the water quality design depth (permanent pool plus extended detention). The width at mid-depth can be found as follows: width = (average top width + average bottom width)/2. *Intent: a long flow path length will improve fine sediment removal.*
- All inlets should enter the first cell. If there are multiple inlets, the length-to-width ratio should be based on the average flow path length for all inlets.
- The minimum freeboard should be 1 foot above the maximum water surface elevation (2 feet preferred) for on-line basins and 1 foot above the maximum water surface elevation for off-line basins.

### *Internal Berms and Baffles*

- The berm or baffle dividing the forebay from the main basin should extend across the full width of the retention basin and be keyed into the basin side slopes. If the berm embankments are greater than 4 feet in height, the berm must be constructed by excavating a key equal to 50% of the embankment cross-sectional width at its base. This requirement may be waived if recommended by a KY licensed civil engineer for the specific site conditions. The geotechnical investigation must consider the situation in which one of the two cells is empty while the other remains full of water.
- The top of the berm should extend to the permanent pool surface. Submerged berm side slopes may be no steeper than 4:1 H:V.
- If good vegetation cover is not established on the berm, erosion control measures should be used to prevent erosion of the berm back-slope when the basin is initially filled or when refilling after drought.

- The interior berm or baffle may be a retaining wall provided that the design is prepared and stamped by a licensed civil engineer. If a baffle or retaining wall is used, it should be submerged one foot below the permanent pool surface to discourage access by pedestrians.

### *Embankments and Side Slopes*

Embankments are earthen slopes or berms used for detaining or redirecting the flow of water. Basin embankments must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed civil engineer in Kentucky) free of loose surface soil materials, roots, and other organic debris. Embankments should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. Side slopes of 4:1 are recommended for slopes facing inward on the wet pond to promote safety and provide berm stability.

### *Water Supply*

- Water balance calculations should be provided to demonstrate that adequate water supply will be present to maintain a pool of water during a drought year when precipitation is 50% of average for the site. Water balance calculations should include evapotranspiration, infiltration, precipitation, spillway discharge, and dry weather flow (where appropriate).
- Where water balance indicates that losses will exceed inputs, a source of water should be provided to maintain the basin water surface elevation throughout the year. The water supply should be of sufficient quantity and quality to not have an adverse impact on the retention basin water quality. Water that meets drinking water standards should be assumed to be of sufficient quality.

### *Liner Considerations*

- If a liner is used to help maintain the permanent pool or to protect groundwater quality (see Site Suitability Considerations), a layer of soil is recommended above the liner to support planned vegetation and protect the liner from damage during maintenance. A landscape architect or botanist should be consulted for further guidance on soil requirements necessary to support the selected vegetation.

### *Water Quality Design Features*

- Retention basins that are located in publicly-accessible or highly visible locations should include design features that will improve and maintain the quality of water within the BMP at a level suitable for the proposed location and uses of the surrounding area. Typical design features include aeration, pumped circulation (provided that the circulation design prevents sediment resuspension), filters, biofilters, and other facilities that operate year-round to remove pollutants and nutrients. Water quality design features will result in higher quality water in the BMP and lower discharges of pollutants downstream.
- Retention basins should have a maintenance plan that includes regular collection and removal of trash from the area within and surrounding the BMP.

### *Energy Dissipation*

- Riprap aprons or other energy dissipation measures must be provided at all inlets. An analysis of backwater effects is required if the inlet will become submerged. Tide gates should be used if backwater is a concern.

- Energy dissipation controls must also be used at the outlet/spillway of the retention basin unless the wetland discharges to a storm water conveyance system or hardened channel.

### Vegetation

Vegetating wet ponds is considered optional. If included, the guidelines below should be adhered to. A plan should be prepared that indicates how aquatic, temporarily submerged areas (land submerged at design volume, but not part of the permanent pool) and terrestrial areas will be stabilized with vegetation. A landscape architect or botanist should be consulted to help identify the most appropriate mix of plants and/or grasses to include in the retention pond while considering the following:

- Emergent aquatic vegetation should cover 25-75% of the area of the permanent pool in a mature basin (e.g., 3-5 years).
- Above the permanent pool, a diverse selection of low growing plants that thrive under the specific site, climatic, and watering conditions should be specified. Native or adapted grasses are preferred because they generally require no fertilizer and limited maintenance, and are more drought resistant than exotic plants.
- If the retention pond is treating runoff from areas where deicing salts are applied, salt tolerant vegetation may be needed.
- Irrigation may be required until vegetation is established.
- No trees or shrubs may be planted within 15 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes or manmade structures. Weeping willow (*Salix babylonica*) should not be planted in or near detention basins.
- Prohibited non-native plant species shall not be used. Refer to the Boone County Zoning Regulations (Landscaping section) for a list of prohibited plant species. Further information on invasive plant species in Kentucky can be found at the Early Detection & Distribution Mapping System (http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky).

### Outlet Structure

- An outlet pipe and outlet structure should be provided to allow for the management of the water surface elevation and permit complete drawdown for maintenance.
- For retention basins that incorporate extended detention, outlet structures should be designed to provide 24 to 36 hour drawdown time for the water quality volume above the permanent pool.
- The basin outlet pipe should be sized, at a minimum, to pass the peak flow for the 10-year storm for off-line basins or the flood control design flow rate for online basins. See SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations for calculating the flows for these two events.
- See the outlet design guidance and the example hydraulic control schematics section in Appendix E and F, respectively, for further information.

### Emergency Spillway

Emergency overflow spillways are intended to control the location of basin overtopping and safely direct overflows back into the downstream conveyance system or other acceptable discharge point. Spillways should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

### *Safety Considerations*

Safety is provided either by fencing of the facility or by managing the contours of the basin to eliminate drop-offs and other hazards. Fencing should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. The design engineer must ensure that the final plans sufficiently protect maintenance crews and the general public from potential hazards associated with the wet pond design.

### *Maintenance Access*

Maintenance access road(s) should be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Access shall be designed in accordance with SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

## DESIGN PROCEDURE

Retention basins should be sized to contain the total water quality design volume plus sediment storage plus the freeboard requirements. Standard grading design should be implemented to estimate excavation and embankment fill quantities necessary while meeting the minimum design requirements described above. Optional methods for sizing outlet structures for meeting the water quality drain time requirements are provided in Appendix E. The recommended procedures for estimating the volume and footprint area of a retention basin are outlined as follows.

### *Step 1: Calculate the Water Quality Design Volume*

The water quality design flow volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

### *Step 2: Calculate Preliminary Geometry Based on Site Constraints*

Determine the active volume of the forebay using the fractional volume ($FV_{fb}$) requirements for the forebay (10-20%). Similarly determine active volume of main cell using the fractional volume ($FV_{mc}$) requirements for the main basin (80-90%)

$$V_{fb} = V_{wq} \frac{FV_{fb}}{100}$$

$$V_{mc} = V_{wq} \frac{FV_{mc}}{100}$$

Where:
$V_{wq}$ = total water quality volume of wet pond ($ft^3$)
$FV_{fb}$ = fractional water quality volume of forebay (10 to 20%)
$FV_{mc}$ = fractional water quality volume of main cell (80 to 90%)
$V_{fb}$ = volume of forebay ($ft^3$)

Calculate surface area of forebay and main cell using average depth of forebay and average depth of main cell.

$$A_{fb} = \frac{V_{fb}}{D_{fb}}$$

$$A_{mc} = \frac{V_{mc}}{D_{mc}}$$

Where:
$A_{fb}$ = Active forebay surface area ($ft^2$)
$A_{mc}$ = Active main cell surface area ($ft^2$)
$V_{fb}$ = volume of forebay ($ft^3$)
$V_{mc}$ = volume of main cell ($ft^3$)
$D_{fb}$ = average depth of forebay (ft)
$D_{mc}$ = average depth of main cell (ft)

Select either a width or length for the facility based on site constraints and the space available and calculate remaining dimensions using the surface areas for the forebay and the main cell.

Calculate the non-active volumes and dimensions of the facility including berms, embankments and space needed for sediment storage. Add the non-active dimensions to the dimensions of the active forebay and main cell components to obtain the foot print dimensions of the facility.

### *Step 3: Select Flow Control Structures and Calculate Outlet Structure Dimensions*

Provide adequate energy dissipation at inlets and size stilling basins as needed to prevent erosion. Recommended methods for sizing outlet structures for meeting the water quality drain time requirements and matching pre-development peak discharges are provided in Appendix E. Emergency spillways should be sized to convey the routed 100-yr design flow rate. Refer to SD1's Storm Water Rules and Regulations or Boone County Subdivision Regulations for acceptable methods for computing flood control design flows.

## SIMPLE WATER BALANCE CALCULATION

A water balance is highly recommended to ensure that the wet pool will not dry out during drought conditions (< 50% of normal precipitation or ~ 1.65 inches per month on average). While this water balance is quite simplified, it should serve as a planning-level guide for determining the need for additional water or a liner. If budget/time permits, a more complete water balance using a continuous hydrologic model is highly recommended.

### *Step 1: Determine the Potential Runoff into the Pond*

$$R = 0.9P(0.05 + 0.90I)\left(\frac{A_{trib}}{A_{pond}}\right)$$

Where:
R = Monthly runoff into the pond (inches of pond depth)
P = Monthly precipitation (use 1.65 inches/month)
I = Fraction of the drainage area (not including pond) that is impervious
$A_{trib}$ = Area that drains to the pond, not including the pond area itself ($ft^2$)
$A_{pond}$ = Area of the pond ($ft^2$)

### *Step 2: Determine the Baseflow to the Pond*

If baseflow measurements have been made, that information can be used as follows:

$$B = 3.154 \times 10^7 \left(\frac{MB}{A_{pond}}\right)$$

Where:
*B* = Baseflow to pond (inches of pond depth per month)
*MB* = Measured baseflow to the pond – assume zero if not measured (cfs)
$A_{pond}$ = Area of the pond ($ft^2$)

### Step 3: Compute the Water Balance

The water balance formula for a wet pond is:

$$P + R + B \geq ET + INF$$

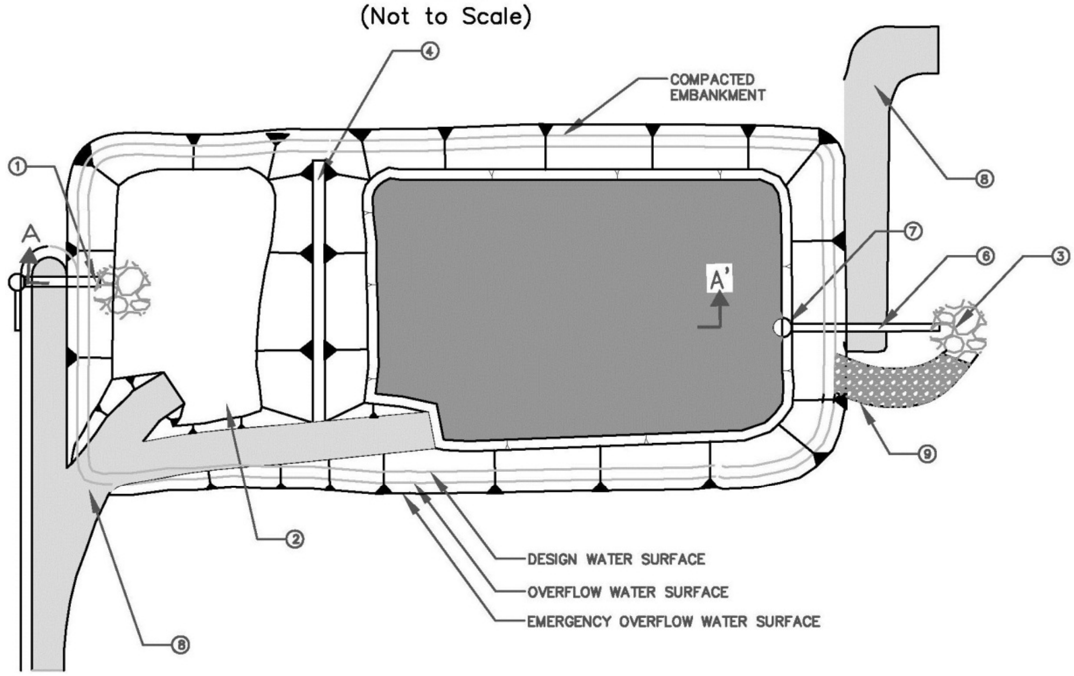
Where:
*P* = Monthly precipitation expected (use 1.65 inches/month for dry conditions)
*R* = Monthly pond depth contributed by runoff from Step 1 (inches/month)
*B* = Monthly baseflow computed in Step 2 (inches/month)
*ET* = Monthly evapotranspiration (a conservative value would be 8 inches/month)
*INF* = Monthly infiltration loss (use measured underlying soil infiltration rate)

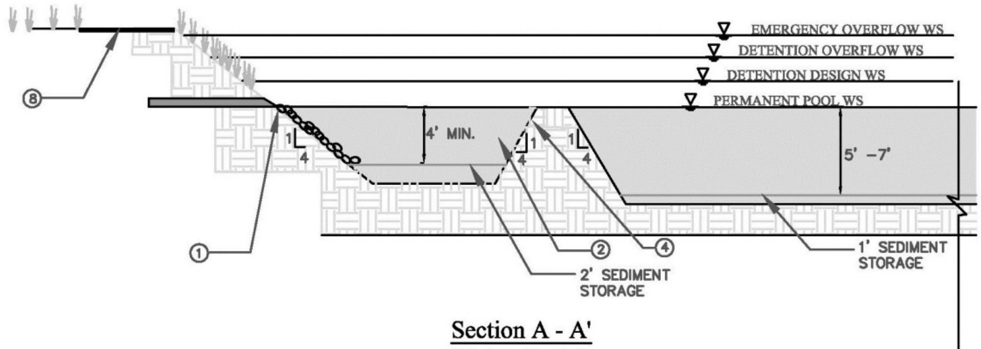
If the inequality in Step 3 is not true (inflow is NOT greater than outflow), then arrangements for a liner and/or an alternate water supply to maintain pond depth in dry times are suggested.

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of retention basins. Other designs are permissible if minimum design criteria are met.

Plan View
(Not to Scale)




Section A - A'
(Not to Scale)

NOTES:

1. RIP RAP APRON OR OTHER ENERGY DISSIPATION SHALL BE PROVIDED AT THE ENTRANCE OF THE INLET PIPE.
2. FOREBAY VOLUME SHALL EQUAL 25% TO 35% OF TOTAL WET POND VOLUME. DEPTH SHALL BE 3' MIN TO 5' MAX PLUS AN ADDITIONAL 2' MIN SEDIMENT STORAGE DEPTH.
3. CONNECT TO DOWNSTREAM STORM DRAIN OR PROVIDE RIP RAP APRON OR OTHER ENERGY DISSIPATION AS NEEDED.
4. BERM SHALL EXTEND ACROSS ENTIRE WIDTH OF THE WET POND.
5. EMERGENT VEGETATION SHALL BE OPTIONALLY PLANTED IN REGIONS OF THE POND THAT ARE 3' DEEP OR LESS.
6. SIZE OUTLET PIPE TO PASS THE ROUTED 100-YR DESIGN FLOW FOR ON-LINE PONDS AND WATER QUALITY PEAK FLOW FOR OFF-LINE PONDS.
7. OUTLET STRUCTURE. IF EXTENDED DETENTION USE MULTI-STAGE RISER OR ORIFICE PLATE.
8. PROVIDE MAINTENANCE ACCESS PER REQUIREMENTS OF PARTY RESPONSIBLE FOR MAINTENANCE.
9. EMERGENCY SPILLWAY SHALL BE SIZED TO PASS THE ROUTED 100-YR DESIGN FLOW.

## MAINTENANCE

Maintenance is of primary importance if retention basins are to continue to function as originally designed. A specific maintenance plan should be formulated for each facility outlining the schedule and scope of maintenance operations, as well as the data handling and reporting requirements. A summary of the routine and major maintenance activities recommended for retention basins is shown in the table below.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Remove trash and debris<br>• Remove evidence of visual contamination from floatables such as oil and grease<br>• Thin vegetation and mow as needed (grass height kept below 9" high)<br>• Eradicate noxious weeds |
| As needed (within 48 hours after every storm greater than 1 inch) | • Clean out sediment from inlets and outlets<br>• Stabilize slopes using erosion control measures (e.g. rock reinforcement, planting of grass, compaction) |
| As needed (infrequently) | • Repair or replace gates, fences, inlet/outlet and flow control structures as needed to maintain full functionality.<br>• If water quality testing shows that anoxic conditions are occurring at the bottom of the pond (usually only a problem in deeper ponds), consider some form of recirculation or aeration, such as a fountain or aerator, to prevent low dissolved oxygen conditions. The aerator should be sized such that mixing does not extend into the sediment storage zone and re-suspend sediments held within the basin. Fountains and aerators are not allowed in wet pools with less than a 5 foot design depth.<br>• Remove dead, diseased, or dying trees or those hindering maintenance.<br>• Replace any missing rock and soil at top of spillway.<br>• Remove forebay sediment when forebay capacity has been decreased by 50%. Remove sediment when six inches have accumulated across main basin bottom.<br>• Repair berm/dike breaches and stabilize eroded parts of the berm<br>• Repair and rebuild spillway as needed to correct severe erosion damage<br>• Install or repair basin liner to ensure that forebay and main basin maintain permanent pools<br>• Correct problems associated with berm settlement<br>• Eliminate noxious weeds, pests, and conditions suitable for creating ideal breeding habitat<br>• Remove algae mats as often as needed to prevent coverage of more than 20% of basin surface<br>• Take photographs before and after maintenance (recommended) |
| Annually | • Verify berms are not settling. Consult a civil engineer to determine the source of settling if the berm is serving as a dam.<br>• Verify there are no discernible water seeps through the berms. Consult a civil engineer to inspect/correct if seeps persist.<br>• Remove any trees or large shrubs growing on downstream side of berms to eliminate habitat for burrowing rodents. |

## ADDITIONAL SOURCES OF INFORMATION


AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001. http://georgiastormwater.com.

Boone County Planning Commision. Boone County Subdivision Regulations. 2010. http://www.boonecountyky.org/pc/2010SubdivisionRegs/2010SubRegs.pdf.

Cahill Associates, Inc. Pennsylvania Stormwater Best Management Practices Manual. 2006.

City of Portland, Oregon. Stormwater Management Manual. 2008. http://www.portlandonline.com/bes/index.cfm?c=47953&

Coastal Georgia Regional Development Center. Green Growth Guidelines. 2006.

Hunt, W. and W. Lord. "Maintenance of Stormwater Wetlands and Wet Ponds." Urban Waterways. North Carolina State University and North Carolina Cooperative Extension. Raliegh, NC., 2006.

Nashville, Tennessee. Stormwater Management Manual, Volume 4. 2009. http://www.nashville.gov/stormwater/regs/SwMgt_ManualVol04_2009.asp

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U.S. EPA, 2006, Stormwater Menu of BMPs: Wet Detention Basin. 4 Nov. 2010. http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm, Office of Water, Washington DC.

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# STORM WATER WETLAND

Structural Best Management Practice

Georgia Storm Water Management Manual

| PERFORMANCE | | | | | |
|---|---|---|---|---|---|
| H | Sediment | | M | Bacteria |
| H | Metals | | H | Trash and debris |
| H | Oil and grease | | L | Volume Reduction |
| M | Nutrients | | M | Peak Flow Control |

H - High, M - Medium, L - Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

Volume Control ☐

Quality Control ☑

*Applications*

- Regional detention & treatment
- Roads, highways, parking lots, commercial, residential
- Parks, open spaces, and golf courses

*Advantages*

✓ Enhanced pollutant removal
✓ Suspended solids and particulate-bound pollutant removal
✓ Aesthetically pleasing
✓ Creates wildlife habitat
✓ Treatment of large tributary areas

*Limitations*

– Supplemental water may be required if water level is to be maintained
– Large footprint area
– Mosquito control may be required
– Management required, including drawdown and invasive removal

STORM WATER WETLAND

## DESCRIPTION

A storm water wetland is a system consisting of a sediment forebay and one or more permanent micro-pools with emergent, semi-emergent and aquatic vegetation covering a significant portion of the basin. Storm water wetlands typically include components such as an inlet with energy dissipation, a sediment forebay for settling out coarse solids and to facilitate maintenance, basins with shallow sections (1 to 2 feet deep) planted with emergent vegetation, deeper areas or micro pools (3 to 5 feet deep), and a water quality outlet structure. The interactions between the incoming storm water runoff, aquatic vegetation, wetland soils, and the associated physical, chemical, and biological unit processes are a fundamental part of storm water wetlands. Therefore, it is critical that dry weather base flows exceed evaporation and infiltration losses to prevent loss of aquatic biota and to avoid stagnation and vector problems. In situations where dry weather flows are inadequate to support the treatment wetland size, an additional source of water may be needed during summer months. Otherwise, the wetland should be sized based on the available base flow and soil characteristics. In addition to water quality treatment, constructed wetlands can be designed for flow control by including extended detention above the permanent pool elevation.

Storm water wetlands are generally designed as plug flow systems where the water already present in the permanent pool is displaced by incoming flows with minimal mixing and no short circuiting. Plug flow describes the hypothetical condition of storm water moving through the wetland in such a way that older “slugs” of water (meaning water that’s been in the wetland for longer) are displaced by incoming slugs of water with little or no mixing in the direction of flow. Short circuiting occurs when quiescent areas or “dead zones” develop in the wetland where pockets of water remain stagnant, causing other volumes to bypass using shorter paths through the basin (e.g., incoming storm water slugs bypass these zones). Water quality benefits are also improved when the permanent wet pool volume is equal to or greater than the water quality volume, resulting in longer residence times.

It is important to note the difference between storm water wetlands and mitigation wetlands that are constructed as part of mitigation requirements. Constructed mitigation wetlands are intended to provide fully functional habitat similar to the habitat impacted and required to be replaced. Storm water wetlands are intended for water quality treatment and, when applicable, flow control. They should be designed to capture and treat pollutants to protect receiving waters, including natural wetlands and other ecologically significant habitat. The accumulation of pollutants in sediment and vegetation of storm water wetlands may impact the health of aquatic biota. As such, periodic sediment and vegetation removal within storm water wetlands will be required.

## SITE SUITABILITY CONSIDERATIONS

Storm water wetlands can be applied to any location where sufficient open space is available at the downstream end of a tributary area and where native soil conditions or sufficient base flows are available to support the wetland vegetation. Storm water wetlands must be designed with the outlet positioned and/or operated in such a way as to maintain a permanent pool of water. In highly permeable soils, the wetland may need to be lined in order for base flows to match or exceed infiltration losses.

Factors that favor the selection of storm water wetlands over other kinds of BMPs include enhanced treatment capability (including dry-weather flow treatment), wildlife enhancement, aesthetics, passive recreation, educational opportunities, and the ability to mitigate large tributary areas. Factors that may limit the use of storm water wetland basins include overly permeable soils and/or non-existent base flows, public acceptance with regard to the potential for vector infestation, and large footprint to tributary area ratios (up to 12% percent of tributary area, dependent on overall imperviousness of the tributary area). Project site topographies, grading, and the relatively shallower nature of storm water wetlands all factor into the practicality of storm water wetlands in some areas. Water level management is required to manage vegetation and sediment.

Considerations for selecting storm water wetlands for a particular site are summarized in the table below and the subsequent text.

| SITE SUITABILITY CONSIDERATIONS FOR STORM WATER WETLANDS | |
|---|---|
| Tributary Area | > 10 Acres (435,600 $ft^2$) and < 10 $mi^2$ |
| Typical BMP area as percentage of tributary area | 5 - 12 percent |
| Proximity to steep sensitive slopes | Wetlands placed near slopes greater than 15 percent or within 200 feet from a hazardous slope or landslide area require a geotechnical investigation |
| Depth to seasonally high groundwater table | This is a site specific issues that may influence site design (cutting/filling). |
| Hydrologic soil group | Any [2] |

1 - Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller in some instances.
2 – “A” Soils may require a pond liner. “B” soils may require infiltration testing to ensure base flows match or exceed losses.

The effectiveness of a storm water wetland is directly related to the contributing land use, the size of the drainage area, the soil type, slope, drainage area imperviousness, proposed vegetation components and management, and the pond dimensions. Natural low points in the topography are well-suited as constructed wetland locations. Additional site suitability recommendations and potential limitations for constructed wetlands are listed below.

- Placement - Storm water wetlands typically are used for treating areas larger than 10 acres and less than 10 square miles. However, smaller drainage areas are possible and “pocket wetlands” with small footprints may be appropriate for some sites. Storm water wetlands require a regular source of base flow if water levels are to be maintained. If base flow is insufficient, supplemental water may be necessary to maintain water levels in the wet pools.

- Soils – Liners should be considered in storm water wetlands implementations in areas with high permeability soils. A water balance assessment should be used to confirm whether a liner is required to keep water in the wetlands (See design section below). The liner will increase the chances of maintaining a permanent pool in the basin and protect groundwater quality. Conduct one test pit or boring per every 2 acres of permanent pool footprint, with a minimum of two per pond. Include information on the soil texture, color, structure, moisture and groundwater indicators, and bedrock type and condition, and identify all by elevation. Liners can be either synthetic materials or imported lower permeability soils (i.e., clays). Wetlands are not recommended in or near karst terrain.

- Development density - The retrofit of storm water wetlands into highly developed areas is sometimes challenging due to the large space requirements needed for effective treatment and storage. New developments can often incorporate constructed wetlands into community parks and dedicated open space and habitat areas.
- Adjacent Land Uses – Comply with all local zoning ordinances.

## DESIGN CRITERIA

The main challenge associated with storm water wetlands is maintaining desired water levels. Additional design parameters can be found in the following table.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Flood control design discharge rate, $Q_{fc}$ | cfs | See SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. |
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 |
| Sediment forebay volume | % | 10-20% of total basin volume |
| Sediment forebay depth | ft | 3 – 5 (without sediment storage)<br>5 – 7 (with sediment storage) |
| Depth of wetland basin | ft | 1-5; variable; see facility geometry section below |
| Freeboard (minimum) | in | 12 (off-line); 12 min and 24 preferred (on-line) |
| Flow path length to width ratio | L:W | 2:1 (min.); 3:1 (preferred) |
| Side slope (maximum) | H:V | 4:1 (H:V) Interior and 2:1 (H:V) Exterior (4:1 maximum if mowed); Introduce as much microtopography as possible. |
| Vegetation Type | -- | Varies. See vegetation section below and Appendix G |
| Vegetation Height | -- | Varies. See vegetation section below |
| Buffer zone (minimum) | ft | 25 |
| Maintenance access ramp width | ft | 16 |

STORM WATER WETLAND

### *Geometry and Size*

In most cases, the storm water wetland permanent pool should be sized to be greater than or equal to the water quality design volume. Additional surcharge storage may be provided above the permanent pool to meet peak discharge requirements. The surcharge portion of the wetland above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see Dry ED Basin factsheet).

- Storm water wetlands should consist of at least two cells including a sediment forebay and a wetland basin.
- The sediment forebay must contain between 10 and 20 percent of the total basin volume.
- The depth of the sediment forebay should be between 5 and 7 feet (including 2 feet for sediment storage).
- Two or more feet of sediment storage should be provided in the sediment forebay.
- The storm water wetland should be designed with a "naturalistic" shape and a range of depths intermixed throughout the wetland basin to a maximum of 5 feet. Microtopography should be incorporated.

| DEPTH RANGE (FEET) | PERCENT BY AREA |
|---|---|
| 0.1 to 1 | 15 |
| 1 to 3 | 55 |
| 3 to 5 | 30 |

- The flow path length-to-width ratio should be a minimum of 2:1, but preferably at least 3:1 or greater.
- The minimum freeboard should be 1 foot above the routed maximum water surface elevation for off-line basins and 2 foot above the routed maximum water surface elevation for on-line basins.

### *Internal Berms and Baffles*

- A berm or baffle should extend across the full width of the constructed wetland and be keyed into the basin side slopes. If the berm embankments are greater than 4 feet in height, the berm must be constructed by excavating a key equal to 50% of the embankment cross-sectional height and width. This requirement may be waived if recommended by a licensed geotechnical engineer for the specific site conditions.
- The top of the berm should be one foot below the permanent pool surface to discourage public access. Submerged berm side slopes may be up to 2:1.
- If good vegetation cover is not established on the berm, erosion control measures should be used to prevent erosion of the berm back-slope when the basin is initially filled or when refilling after a drought.
- The interior berm or baffle may be a retaining wall provided that the design is prepared and stamped by a licensed civil engineer. If a baffle or retaining wall is used, it should be submerged one foot below the permanent pool surface to discourage access by pedestrians.



### *Embankments and Side Slopes*

Embankments are earthen slopes or berms used for detaining or redirecting the flow of water. Basin embankments must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed civil engineer in Kentucky) free of loose surface soil materials, roots, and other organic debris. Embankments shall be designed in accordance with the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. Side slopes of 4:1 are recommended for slopes facing inward on the wetland to promote safety and provide berm stability.

### *Water Supply*

- Water balance calculations should be provided to demonstrate that adequate water supply will be present to maintain a permanent pool of water during a drought year when precipitation is 50% of average for the site. Water balance calculations should include evapotranspiration, infiltration, precipitation, spillway discharge, and dry weather flow (where appropriate) (see Design Procedure section).
- Where water balance indicates that losses will exceed inputs, a source of water should be provided to maintain the wetland water surface elevation throughout the year. The water supply should be of sufficient quantity and quality to not have an adverse impact on the wetland water quality. Water that meets drinking water standards should be assumed to be of sufficient quality.

### *Soils Considerations*

Implementation of storm water wetlands in areas with highly permeable soils requires liners to increase the chances of maintaining permanent pools and/or micro-pools in the basin. Liners can be either synthetic materials or imported lower permeability soils (i.e., clays). The water balance assessment should determine whether a liner is required. The following conditions can be used as a guideline.

- The forebay of the wetland basin must retain water for at least 10 months of the year.
- The sediment forebay must retain at least 3 feet of water year-round. Local regulations should be considered for other situations requiring a liner such as depth to seasonally high groundwater, depth to bedrock, etc.
- Many wetland plants can adapt to periods of summer drought, so a limited drought period is allowed in the wetland basin. This may allow for a soil liner rather than a geosynthetic liner. The sediment forebay must retain 3 feet of water year-round for presettling to be effective.

- If a liner is used, 1.5 to 2 feet of amended soil cover is recommended to protect the liner and promote vegetation establishment.
- Reuse of onsite hydric soils is recommended if available.

### *Energy Dissipation*

- Riprap aprons or other energy dissipation measures must be provided at all inlets. An analysis of backwater effects is required if the inlet will become submerged. Tide gates should be used if backwater is a concern.
- Energy dissipation controls must also be used at the outlet/spillway of the storm water wetland unless the wetland discharges to a storm water conveyance system or hardened channel.

### *Vegetation*

- The wetland cell(s) should be planted with emergent wetland plants following the recommendations of a wetlands specialist. A mature storm water wetland should have 75% or more vegetative coverage in areas that are less than 3-feet deep.
- Landscaping outside of the basin is required for all constructed wetlands and must adhere to the following criteria so as not to hinder maintenance operations: No trees or shrubs may be planted within 15 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes or manmade structures. Weeping willow (*Salix babylonica*) should not be planted in or near storm water wetlands.
- See Appendix C for a recommended native plant list for storm water wetlands. The plant list should be used as a guide only and should not replace project-specific planting recommendations provided by a wetland ecologist or a landscape architect including recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation establishment and growth.
- Prohibited non-native plant species will not be permitted. Refer to the Boone County Zoning Regulations (Landscaping section) for a list of prohibited plant species. Further information on invasive plant species in Kentucky can be found at the Early Detection & Distribution Mapping System (http:/www.eddmaps.org/tools/stateplants.cfm?id=us_ky).

### *Outlet Structure*

- An outlet pipe and outlet structure should be provided to allow for the management of the water surface elevation and permit complete drawdown for maintenance.
- For wetlands with detention storage, the outlet structure(s) should be designed to provide the required flow attenuation necessary for achieving the peak runoff discharge requirements.
- The wetland outlet pipe should be sized, at a minimum, to pass the water quality design peak flow for off-line basins or flows greater than the peak runoff discharge rate for the routed 100-year design storm for on-line basins.
- See the outlet design guidance and the example hydraulic control schematics section in Appendix E and F, respectively, for further information.

STORM WATER WETLAND

### Emergency Spillway

Emergency overflow spillways are intended to control the location of basin overtopping and safely direct overflows back into the downstream conveyance system or other acceptable discharge point. Spillways should meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

### Safety Considerations

Safety is provided either by fencing the facility or by managing the contours of the basin to eliminate drop-offs and other hazards. Fencing shall meet the requirements of SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. The design engineer must ensure that the final plans sufficiently protect maintenance crews and the general public from potential hazards associated with the wetland design.

### Maintenance Access

Maintenance access road(s) should be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Access shall be designed in accordance with SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

## DESIGN PROCEDURE

Storm water wetlands should be sized to contain the total design volume plus sediment storage plus the freeboard requirements. Standard grading design should be implemented to estimate excavation and embankment fill quantities necessary while meeting the minimum design requirements described above. The recommended procedures for estimating the volume and footprint area of a storm water wetland are outlined as follows.

### *Step 1: Water Quality Design Volume*

The water quality design flow volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

### *Step 2: Preliminary Geometry Based on Site Constraints*

Determine the active volume of the forebay using the fractional volume ($FV_{fb}$) requirements for the forebay (10-20%). Similarly determine active volume of main cell using the fractional volume ($FV_{mc}$) requirements for the main basin (80-90%)

$$V_{fb} = V_{wq} \frac{FV_{fb}}{100}$$

$$V_{mc} = V_{wq} \frac{FV_{mc}}{100}$$

Where:
$V_{wq}$ = total water quality volume of wetland (ft$^3$)
$FV_{fb}$ = fractional water quality volume of forebay (10 to 20%)
$FV_{mc}$ = fractional water quality volume of main cell (10 to 20%)
$V_{fb}$ = volume of forebay (ft$^3$)

Calculate surface area of forebay and main cell using average depth of forebay and average depth of main cell.

$$A_{fb} = \frac{V_{fb}}{D_{fb}}$$

$$A_{mc} = \frac{V_{mc}}{D_{mc}}$$

Where:
$A_{fb}$ = Active forebay surface area (ft$^2$)
$A_{mc}$ = Active main cell surface area (ft$^2$)
$V_{fb}$ = volume of forebay (ft$^3$)
$V_{mc}$ = volume of main cell (ft$^3$)
$D_{fb}$ = average depth of forebay (ft)
$D_{mc}$ = average depth of main cell (ft)

Select either a width or length for the facility based on site constraints and the space available and calculate remaining dimensions using the surface areas for the forebay and the main cell. For the main cell, calculate volumes, surface areas and dimensions for the shallow ($V_{shallow}$ , $A_{shallow}$), deep ($V_{deep}$ , $A_{deep}$), and micro-pool regions ($V_{pool}$ , $A_{pool}$) using the volume distribution shown in the table below such that:

STORM WATER WETLAND

$$V_{mc} = V_{shallow} + V_{deep} + V_{pool}$$

$$A_{mc} = A_{shallow} + A_{deep} + A_{pool}$$

Where:
$V_{shallow}$ = Volume of shallow region of main cell ($ft^3$)
$V_{deep}$ = Volume of deep region of main cell ($ft^3$)
$V_{pool}$ = Volume of micro-pool region of main cell ($ft^2$)
$A_{shallow}$ = Surface area of shallow region of main cell ($ft^2$)
$A_{deep}$ = Surface area of deep region of main cell ($ft^2$)
$A_{pool}$ = Surface area of micro-pool region of main cell ($ft^2$)

| MAIN CELL REGION | DEPTH RANGE (FEET) | PERCENT BY AREA |
|---|---|---|
| Shallow | 0.1 to 1 | 15 |
| Deep | 1 to 3 | 55 |
| Micro-Pool | 3 to 5 | 30 |

Calculate the non-active volumes and dimensions of the facility including berms, embankments and space needed for sediment storage. Add the non-active dimensions to the dimensions of the active forebay and main cell components to obtain the foot print dimensions of the facility.

### *Step 3: Inlets and Outlet Structures*

Provide adequate energy dissipation at inlets and sizing stilling basins as needed to prevent erosion. Various outlet structure design examples are provided in Appendix F. Recommended methods for sizing outlet structures for meeting the water quality drain time requirements and matching pre-development peak discharges are provided in Appendix E. Emergency spillways should be sized to convey the routed 100-yr design flow rate. Refer SD1's Storm Water Rules and Regulations or Boone County Subdivision Regulations for acceptable methods for computing flood control design flows.

## SIMPLE WATER BALANCE CALCULATION

A water balance is highly recommended to ensure that the wet pool will not dry out during drought conditions (< 50% of normal precipitation or ~ 1.65 inches per month on average). While this water balance is quite simplified, it should serve as a planning-level guide for determining the need for additional water or a liner. If budget/time permits, a more complete water balance using of a continuous hydrologic model is highly recommended.

### *Step 1: Determine the potential runoff into the pond*

$$R = 0.9P(0.05 + 0.90I)\left(\frac{A_{trib}}{A_{pond}}\right)$$

Where:
$R$ = Monthly runoff into the pond (inches of pond depth)
$P$ = Monthly precipitation (use 1.65 inches/month)
$I$ = Fraction of the drainage area (not including pond) that is impervious
$A_{trib}$ = Area that drains to the pond, not including the pond area itself ($ft^2$)
$A_{pond}$ = Area of the pond ($ft^2$)

### *Step 2: Determine the baseflow to the pond*

If baseflow measurements have been made, that information can be used as follows:

$$B = 3.154 \times 10^{7} \left( \frac{MB}{A_{pond}} \right)$$

Where:
*B* = Baseflow to pond (inches of pond depth per month)
*MB* = Measured baseflow to the pond (cfs)
$A_{pond}$ = Area of the pond ($ft^2$)

### *Step 3: Compute the water balance*

The water balance formula for a wet pond is:

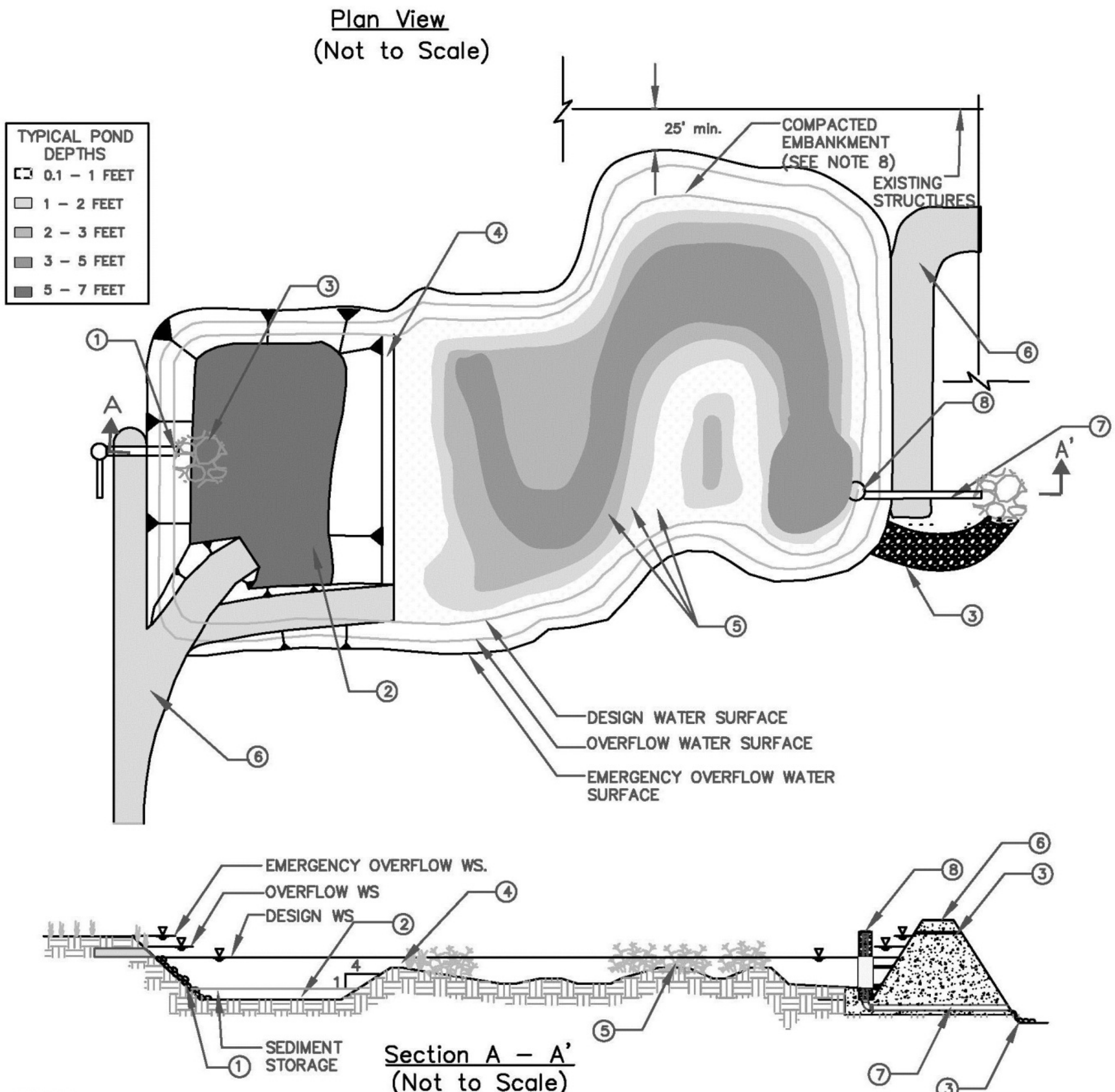
$$P + R + B \geq ET + INF$$

Where:
*P* = Monthly precipitation expected (use 1.65 inches/month for dry conditions)
*R* = Monthly pond depth contributed by runoff from Step 1 (inches/month)
*B* = Monthly baseflow computed in Step 2 (inches/month)
*ET* = Monthly evapotranspiration (a conservative value would be 8 inches/month)
*INF* = Monthly infiltration loss (use measured underlying soil infiltration rate)

If the inequality in Step 3 is not true (inflow is NOT greater than outflow), then arrangements for a liner and/or an alternate water supply to maintain pond depth in dry times are required.

## DESIGN SCHEMATICS

The following schematic should be used as further guidance for design of constructed wetlands. Other designs are permissible if minimum design criteria are met.




NOTES:

1. RIP RAP APRON OR OTHER ENERGY DISSIPATION SHALL BE PROVIDED AT THE ENTRANCE OF THE INLET PIPE.
2. SEDIMENT FOREBAY. FOREBAY VOLUME SHALL EQUAL 10% TO 20% OF TOTAL WETLAND VOLUME. FOREBAY DEPTH SHALL BE 3' MIN TO 5' MAX PLUS AN ADDITIONAL 2' MIN SEDIMENT STORAGE DEPTH.
3. INSTALL EMERGENCY SPILLWAY AS NEEDED.
4. BERM SUBMERGED 1' BELOW DESIGN WATER SURFACE ELEVATION. EXTEND BERM ACROSS ENTIRE WIDTH OF THE WETLAND.
5. WETLAND VEGETATION. PLANTING SCHEME MUST BE DESIGNED BY A WETLAND SPECIALIST.
6. PROVIDE MAINTENANCE ACCESS PER REQUIREMENTS OF PARTY RESPONSIBLE FOR MAINTENANCE.
7. SIZE OUTLET PIPE TO PASS THE ROUTED 100-YEAR DESIGN FLOW FOR ONLINE AND WATER QUALITY PEAK FLOW FOR OFFLINE BASINS.
8. OUTLET STRUCTURE. IF EXTENDED DETENTION USE MULTI-STAGE RISER OR ORIFICE PLATE.

# MAINTENANCE

## *General Requirements*

Maintenance is of primary importance if storm water wetlands are to continue to function as originally designed. A specific maintenance plan should be formulated for each facility outlining the schedule and scope of maintenance operations, as well as the data handling and reporting requirements. A summary of the routine and major maintenance activities recommended for wetland basins is shown in the table below.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Remove trash and debris<br>• Remove evidence of visual contamination from floatables such as oil and grease<br>• Thin vegetation and mow as needed (grass height kept below 9" high)<br>• Eradicate noxious weeds (upland buffer, wetland, and aquatic) |
| As needed (within 48 hours after every storm greater than 1 inch) | • Clean out sediment from inlets and outlets<br>• Stabilize slopes using erosion control measures (e.g. rock reinforcement, planting of grass, compaction) |
| As needed (infrequently) | • Repair or replace gates, fences, inlet/outlet and flow control structures as needed to maintain full functionality.<br>• Remove dead, diseased, or dying trees or those hindering maintenance activities.<br>• Replace any missing rock and soil at top of spillway.<br>• Remove forebay sediment when forebay capacity has been decreased by 50%. Remove sediment when 1 foot has accumulated across main basin bottom.<br>• Repair berm/dike breaches and stabilize eroded parts of the berm<br>• Repair and rebuild spillway as needed to correct severe erosion damage<br>• Install or repair basin liner to ensure that forebay and main basin maintain permanent pools<br>• Correct problems associated with berm settlement<br>• Eliminate noxious weeds, pests, and conditions suitable for creating ideal breeding habitat<br>• Remove algae mats as often as needed to prevent coverage of more than 20% of basin surface<br>• Take photographs before and after maintenance (recommended) |
| Annually | • Verify berms are not settling. Consult a civil engineer to determine the source of settling if the berm is serving as a dam.<br>• Verify there are no discernible water seeps through the berms. Consult a civil engineer to inspect/correct if seeps persist.<br>• Remove any trees or large shrubs growing on downstream side of berms to eliminate habitat for burrowing rodents.<br>• Exercise water management devices during inspections and vegetation management. |

STORM WATER WETLAND

## ADDITIONAL SOURCES OF INFORMATION


AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001.

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# STREET TREES

Structural Best Management Practice

STREET TREES

Honeylocust – Photo from http://www.dnr.state.oh.us/Home/urban/ostep/cd4150_99100/tabid/5521/Default.aspx

| PERFORMANCE | | | | | |
|---|---|---|---|---|---|
| NA | Sediment | | NA | Bacteria |
| NA | Metals | | NA | Trash and debris |
| NA | Oil and grease | | L | Volume Reduction |
| NA | Nutrients | | L | Peak Flow Control |

NA – Not Applicable

H – High, M – Medium, L – Low

Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

* Street trees are hydrologic source controls where water quality benefits are limited to the runoff volume reduced.

Volume Control ☑

Quality Control ☐

*Applications*

- Can be incorporated in residential/ commercial green streets
- Can be used along sidewalks, streets, parking lots, or driveways

*Advantages*

✓ Volume & peak flow reduction

✓ Runoff temperature reduction

✓ Shading from tree reduces urban heat island effect

✓ Raise property values through enhanced aesthetics and greater biodiversity.

*Limitations*

– Need sufficient space for root system and tree canopy

– Must be sited in a location which provides adequate sunlight

– Tree species selection must meet the physical and environmental characteristics of the site

– Does not provide water quality treatment

## DESCRIPTION

Street trees can provide several storm water benefits by intercepting rainfall, including peak flow control, increased infiltration and evapotranspiration, and runoff temperature reduction. These benefits are most measurable for storms of less than 0.5 inch over 24 hours, therefore street trees alone will not meet the water quality or storm water reduction requirements. Although deciduous trees are not as effective during winter months, evergreen trees are effective year round for these smaller storms and portions of larger storms. Generally, large trees with small leaves are the most efficient rainfall interceptors. The volume of precipitation intercepted by the canopy reduces the runoff volume downstream treatment BMPs must mitigate. Shading provided by the tree reduces the heat island effect as well as the temperature of adjacent impervious surfaces, over which storm water flows, and thus reduces the heat transferred to downstream receiving waters. Tree roots also strengthen the soil structure and provide infiltrative pathways, simultaneously reducing erosion potential and enhancing infiltration.

STREET TREES

## SITE SUITABILITY CONSIDERATIONS

Street trees can be used near impervious surfaces provided there is sufficient room for both the canopy and the root system. Site suitability issues include:

- Placement – Placement of street trees should consider space requirements and environmental factors. A 20 to 30 foot diameter canopy (at maturity) is recommended for storm water mitigation purposes. The canopy must be accommodated within the space available. The tree canopy and root system should not interfere with subsurface utilities, suspended power lines, or buildings and foundations. Required setbacks should be adhered to and infiltration through the trees should not lead to geotechnical hazards related to adjacent structures. In addition to infrastructure concerns, public safety concerns must be considered. For instance, trees should not impede pedestrian and vehicle sight lines or be planted too close to walks and drives.
- Development density – Street trees can be easily incorporated into denser developments next to streets, sidewalks, and driveways, provided the placement requirements above are met.
- Adjacent Land Uses – Street trees can typically be incorporated into residential, commercial, multi-family, and institutional land uses with ease. For freeway and industrial areas, opportunities for placement of street trees may be more limited due to safety and clearance requirements.
- Physical and Environmental Factors – Environmental factors must be considered so the full potential of the trees can be achieved. It is critical that soil types, soil pH, moisture, sunlight, exposure and wind be matched to the tree species selected.

## DESIGN CRITERIA

The following sections describe basic design guidance for street trees.

### *Geometry and Size*

- Street trees should be planted with ample room for the roots to spread. The amount of space that roots will require and the direction of root growth will depend on the tree species used. Power lines should also be kept in mind when considering future growth potential.

American Linden - Photo from http://www.coloradotrees.org/treeomonth/2002/apr_02.htm

STREET TREES

### *Planting Media*

- Soils should be preserved in their natural condition (if appropriate for planting) or modified with engineered bioretention soil as described in Appendix B. If unusual site conditions are encountered, a landscape architect or plant biologist should be consulted.

### *Tree Species*

- The retention of existing healthy, substantial trees should occur wherever possible.
- Prohibited non-native species shall not be used. Refer to the Boone County Zoning Regulations (Landscaping section) for a list of prohibited plant species. Further information on invasive plant species in Kentucky can be found at the Early Detection & Distribution Mapping System (http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky)
- A street tree selection guide, such as the Virginia Urban Street Tree Selector (http://dendro.cnre.vt.edu/treeselector/index.cfm), may be of assistance in selecting species appropriate for the site design constraints (e.g., parkway size, tree height, canopy spread, etc.). The tree type selected should not require long-term irrigation.
- Selection of trees should consider fruit/acorns/leaves/etc. produced by the tree, as this will affect maintenance requirements.
- A variety of species should be incorporated into any street tree program to improve the diversity, enhance the aesthetics, and reduce the risk from insects/disease.
- All tree planting shall comply with rules set out in the Boone County Zoning Regulations (Landscaping section). Planting lists can also be found in this document.

STREET TREES

## MAINTENANCE

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Inspect trees for branching structure, damage caused by storms, decay, insects, and disease.<br>• Water trees until roots are established. |
| As needed (infrequently) | • Prune trees to maintain good form and proper distance from man-made structures. Do not dead head or top trees.<br>• Replace any diseased or dying trees.<br>• Sweep streets to remove any fruit/acorns/leaves.<br>• Tree trimming should be completed by a certified arborist. |

## ADDITIONAL SOURCES OF INFORMATION

AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001.

Boone County Planning Commission. Boone County Subdivision Regulations. 2010. http://www.boonecountyky.org/pc/2010SubdivisionRegs/2010SubRegs.pdf.

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Virginia Tech Forestry Department. Virginia Urban Street Tree Selector. http://dendro.cnre.vt.edu/treeselector/index.cfm

# SUBSURFACE VAULT

Structural Best Management Practice

SUBSURFACE VAULT

ChamberMaxx®, produced by CONTECH (Picture and information at http://www.contech-cpi.com/Products/Stormwater-Management/Detention-and-Infiltration/ChamberMaxx.aspx)

Volume Control ☑

Quality Control ☐

| PERFORMANCE | | | |
|---|---|---|---|
| M | Sediment | UNK | Bacteria |
| UNK | Metals | UNK | Trash and debris |
| UNK | Oil and grease | M | Volume Reduction |
| UNK | Nutrients | M | Peak Flow Control |

UNK – Unknown; depends on design components.
H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

*Applications*

- Roads and parking lots
- Parks and recreation
- Single and multi-family residential
- Commercial and mixed use
- Below permeable pavement or bioretention facilities

*Advantages*

✓ Appropriate for sites with limited surface space
✓ Can be installed below roads, parking lots, parks, and athletic fields
✓ Provides peak rate control

*Limitations*

- Not appropriate for high-pollutant land-uses
- Require pretreatment
- If placed underneath roads, parking lots, etc., structural integrity must be adequate to support loads above

## DESCRIPTION

Subsurface vaults are underground structures that are, in many ways, similar to above ground detention or retention basins. More expensive than above ground facilities, they are used primarily in ultra-urban areas where land values are high and very little pervious space exists to implement more traditional surface BMPs. Consequently, they must be designed to account for loading due to parking or building uses above them. Most often constructed of plastic or concrete, they provide temporary storage of storm water and can be designed with open bottoms to allow infiltration or a wet pool to provide sedimentation. A number of vendors offer proprietary subsurface storage and infiltration products that can be used in a variety of applications and configurations. The calculations below assume this vault would be used for treatment of the water quality volume through infiltration through an open bottomed vault system.

SUBSURFACE VAULT

## SITE SUITABILITY CONSIDERATIONS

Subsurface vaults can be used for peak reduction, infiltration, and sedimentation of runoff from a number of different land uses depending on the configuration. These facilities may also be placed below permeable pavement or bioretention areas to increase the subsurface storage volume of these facilities. Due to the higher cost associated with subsurface storage, subsurface vaults are generally only considered for relatively small drainage areas (<5 acres). However, any size drainage area is possible given adequate subsurface space is available. Other site suitability considerations are listed below.

| SITE SUITABILITY CONSIDERATIONS FOR SUBSURFACE VAULTS | |
|---|---|
| Tributary Area[1] | < 5 acres; 217,800 $ft^2$ |
| Proximity to steep sensitive slopes | Only non-infiltrating vaults allowed on slopes steeper than 15% or within 50 feet of a steep slope or landslide hazard area. Additionally, a geotechnical investigation should be performed. None of the systems are allowed on slopes steeper than 20%. |
| Proximity to private water sources[2] | NA due to ultra-urban nature of BMP. |
| Depth to seasonally high groundwater table below subsurface vault system bottom | > 10 ft if designed for infiltration |
| Hydrologic soil group | Any; A or B if designed for infiltration |

1 – Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger in some instances.

2 – Public wells are governed by wellhead protection programs (A GIS layer showing protection program areas is available at http://kygisserver.ky.gov/geoportal/catalog/search/viewMetadataDetails.page?uuid=%7BEAE876B0-FBD0-4362-A7CA-75DA3B12BAA8%7D). Contact the Wellhead Protection Program Coordinator at the Kentucky Division of Water, Groundwater Branch for more information.

Subsurface vaults designed as infiltration BMPs should be sited only where infiltration is appropriate. Pre-treatment structures or BMPs are required to provide removal of coarse solids prior to infiltration. Water bypassing pre-treatment cannot be directed towards the subsurface vault. Other site suitability considerations are included below.

- Placement – Subsurface vaults can be sited below roads and parking lots, thus they require little in the way of surface space for the detention system itself. However, pre-treatment BMPs, such as hydrodynamic separators or baffle boxes are required for all subsurface vault facilities, so space requirements for pre-treatment BMPs should be considered prior to siting a subsurface vault facility. Facilities beneath roads and parking areas must meet H-20 load requirements.

- Development density – Subsurface vault facilities are a good option for dense developments because they can be sited below roads, parking lots, parks, and athletic fields.

- Adjacent Land Uses – Subsurface vault facilities can be used for mixed-use, commercial, single-family, multi-family, roads and parking lots, and parks and open space land uses. Pre-treatment must be provided to remove sediment and filter out pollutants. Subsurface infiltration vaults are not recommended to treat active construction sites or other areas high sediment loading.

- Geotechnical Considerations – Subsurface vaults should not be located in areas with known geotechnical hazards (including landslides, liquefaction zones, steep slopes, etc.). Required set-backs from foundations, structures and utilities should be observed.

- Soil type – Subsurface vaults, if designed for infiltration, should only be located where underlying soils are classified as A or B type soils and the design infiltration rate is greater than 0.5 inches per hour (2 in/hr measured). If the measured infiltration rate is less than than 2 in/hr, an underdrain connected to an outlet control structure is recommended (see Design Criteria below).
- Depth Requirements – Depth to groundwater, bedrock, or low-permeability soil layers should be at least 5 feet from the bottom of the facility to ensure that it will completely drain between storms and that infiltrating water will receive adequate treatment through the soils before it reaches the groundwater table.
- Soil or Groundwater Contamination – Subsurface vaults should not be located above areas with known groundwater or soil contamination, as infiltration may result in spreading sub-surface contamination.

## DESIGN CRITERIA

Subsurface vaults can be effective at reducing runoff volumes when soil conditions are amenable to infiltration. If soil conditions are not amenable to infiltration, subsurface vaults are primarily used for coarse sediment removal and peak flow control. In these situations, an underdrain with outlet control structure is recommended to ensure adequate settling time and flow attenuation. To improve infiltration rates of native soils, the top 1-2 feet of the infiltrating surface should be amended at a rate of 2 parts native soils to 1 part coarse sand. The following table summarizes the minimum design criteria for underground vaults. Additional sizing criteria and design guidance are provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Water quality design volume, $V_{wq}$ | $ft^3$ | See Chapter 3 for instructions on calculating $V_{wq}$ |
| Drawdown time | hr | 48 (maximum) if infiltration only (no underdrain)<br>36-48 if detention w/ underdrain |
| Other sizing parameters | -- | Refer to manufacturer guidelines |
| Pre-treatment | -- | Required |

### *Pretreatment*

Pretreatment is required for proprietary subsurface BMPs in order to reduce the sediment load entering the facility and maintain the infiltration rate of the facility. Pretreatment refers to design features that provide settling of sediment particles before runoff reaches a storm water best management practice. This eases the long-term maintenance burden and potential of failure. To ensure that pretreatment mechanisms are effective, designers should incorporate sediment reduction BMPs as pre-treatment. Sediment reduction BMPs may include vegetated swales, vegetated filter strips, sedimentation basins, sedimentation manholes and hydrodynamic separation devices. The use of at least two pretreatment devices is recommended for infiltration BMPs.

### *Sizing*

- Proprietary subsurface BMPs shall be sized to capture the entire storm water quality design volume $V_{wq}$. See Chapter 3 for further detail in calculating this.
- To provide adequate treatment, the stored water must be either infiltrated or detained for at least 36 hours. Stored water should drain in no more than 48 hours so the storage capacity is regenerated prior to incoming storms.
- Depending on the design and orientation of the subsurface facility with respect to the downstream conveyance, a multi-stage outlet structure may be used to achieve peak flow control. Refer to manufacturer's information for outlet options or see Appendix E for some example outlet structure designs. An underdrain may be used and connected to the outlet control structure to ensure complete drawdown of the stored volume.
- The percolation rate will decline as particulates accumulate in the infiltrative layer. It is important that adequate conservatism is incorporated in the selection of design percolation rates. An in-situ infiltration test is required for subsurface infiltration facilities at the bottom of the facility or at the top of a confining layer.
- For the sizing guidelines, refer to the manufacturer's guidance. If no underdrains are present to ensure complete drawdown, an observation well extending at least 2 feet into native soil below the facility is recommended to assist with identifying drainage problems.

### *Underdrains*

- If underdrains are required, then they must be made of perforated or slotted, polyvinyl chloride (PVC) pipe conforming to ASTM D 3034 or equivalent or corrugated high density polyethylene (HDPE) pipe conforming to AASHTO 252M or equivalent. Underdrains shall slope at a minimum of 0.5 percent, and smooth and rigid PVC pipes shall be used as underdrains with slopes of less than 2 percent.
- The perforations or slots shall be sized to prevent the migration of the drain rock into the pipes, and shall be spaced such that the pipe has a minimum of 1 square inch of opening per lineal foot of pipe.
- The underdrain pipe must have a 6-inch minimum diameter, so it can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe must be placed at the terminal ends of the underdrain. The cleanout risers shall be plugged with a lockable well cap. It is recommended to keep the cap locked in areas prone to vandalism.
- The underdrain shall be bedded with 6 inches of drain rock and backfilled with a minimum of 6 inches of drain rock around the top and sides of the underdrain. The drain rock shall consist of clean, washed No. 57 stone, conforming to the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet, or an approved equal, that meets the gradation requirements listed in the table below.

| SIEVE SIZE | PERCENT PASSING |
|---|---|
| 1-$^1/_2$ inch | 100 |
| 1 inch | 95-100 |
| $^1/_2$ inch | 25-60 |
| US No. 4 | 0-10 |
| US No. 8 | 0-5 |

- The drain rock must be separated from the native soil layer below and to the sides with an approved non-woven geotextile fabric. The non-woven geotextile filter fabric should have a minimum flow rate of 50 gal/min/ft$^2$. Unless otherwise approved, the non-woven geotextile fabric shall conform to the Type II Fabric Geotextiles for Underdrains described in the Standard Specifications for Road and Bridge Construction published by the Kentucky Transportation Cabinet. The minimum requirements for the non-woven geotextile filter fabric are provided below:

| GEOTEXTILE PROPERTY | VALUE | TEST METHOD |
|---|---|---|
| Grab Strength (lbs.) | 80 | ASTM D4632 |
| Sewn Seam Strength (lbs.) | 70 | ASTM D4632 |
| Puncture Strength (lbs.) | 25 | ASTM D4833 |
| Trapezoid Tear (lbs.) | 25 | ASTM D4533 |
| Apparent Opening Size US Std. Sieve | No. 50 | ASTM D4751 |
| Permeability (cm/s) | 0.010 | ASTM D4491 |
| UV Degration at 150 hrs. | 70% | ASTM D4355 |
| Flow Rate (gpm/ft$^2$) | 50 | ASTM D4491 |

- The underdrain pipe must drain freely to an acceptable discharge point.

## DESIGN PROCEDURE AND SCHEMATICS

Refer to manufacturer's information for design procedures and schematics specific to their product. For subsurface infiltration facilities, they must be designed to completely drain within 48 hours. For subsurface detention facilities, they must be designed to discharge in 36 to 48 hours.

### *Step 1: Design Volume*

The water quality design volume, $V_{wq}$, shall be determined using the procedure provided in Chapter 3.

### *Step 2: Design Infiltration Rate*

The design infiltration rate is based on the hydraulic conductivity of the native soil as determined using an in-situ percolation test measured at the elevation of the proposed bottom of the facility or at the depth of a limiting layer multiplied by a factor of safety of 0.25:

$$k_{native} = 0.25 \cdot k_{measured}$$

Where:

$k_{native}$ = the design infiltration rate for the native soils (in/hr)
$k_{measured}$ = the measured infiltration rate (in/hr)

If $k_{native}$ is less than 0.5 in/hr, then an underdrain connected to an outlet control structure is recommended (skip to Step 4).

### *Step 3: Infiltrating Surface Area*

The surface area computed here represents the open area at the bottom of the subsurface vault:

$$A = \frac{12 \cdot V_{wq}}{t \cdot k_{native}}$$

Where:

$A$ = surface area at the bottom of the subsurface vault ($ft^2$)
$V_{wq}$ = water quality design volume ($ft^3$)
$k_{native}$ = design infiltration rate of the native soil (in/hr)
$t$ = target drain time (hrs) [use 48 hours or less]

### *Step 4: Select Flow Control Structures and Calculate Outlet Structure Dimensions*

Recommended methods for sizing outlet structures for meeting the water quality drain time requirements and matching pre-development peak discharges are provided in Appendix E. Refer to SD1's Storm Water Rules and Regulations or Boone County's Design Standards for Subdivision Regulation for acceptable methods for computing flood control design flows.

## MAINTENANCE

Refer to manufacturer instructions for maintenance procedures and frequency. Routine maintenance will probably include removal of trash, debris, and sediment at inlets/outlets, and inspections to ensure facility is draining within the required time and to ensure there is no mosquito breeding occurring near the facility. Some manufacturers provide maintenance packages as well. Follow all applicable confined space entry procedures when performing maintenance.

## ADDITIONAL SOURCES OF INFORMATION

The mention of trade names or commercial products below does not constitute endorsement or recommendation for use by SD1 or the City of Florence.

SUBSURFACE VAULT MANUFACTURER WEBSITES

| DEVICE | MANUFACTURER | WEBSITE |
|---|---|---|
| A-2000™ | Contech® Construction Products Inc. | www.contech-cpi.com/stormwater/13 |
| ChamberMaxx™ | Contech® Construction Products Inc. | www.contech-cpi.com/stormwater/13 |
| CON/SPAN Vaults™ | Contech® Construction Products Inc. | www.contech-cpi.com/stormwater/13 |
| CON/Storm™ | Contech® Construction Products Inc. | www.contech-cpi.com/stormwater/13 |
| Perforated Corrugated Metal Pipe (CMP) | Contech® Construction Products Inc. | www.contech-cpi.com/stormwater/13 |
| Drywell StormFilter | Contech® Construction Products Inc. | www.contech-cpi.com/stormwater/13 |
| CUDO® Water Storage System | KriStar Enterprises Inc. | www.kristar.com |
| D-Raintank® Matrix Tank Modules | Atlantis® | www.atlantis-america.com |
| EcoRain™ Modular Rain Tank | EcoRain Systems Inc. | www.ecorain.com |
| Landmax® | Hancor® | www.hancor.com |
| Landsaver™ | Hancor® | www.hancor.com |
| Rainstore[3] | Invisible Structures Inc. | www.invisiblestructures.com |
| StormChambers™ | Hydrologic Solutions, Inc. | www.hydrologicsolutions.com |
| Stormtech® SC-740 and SC-310 Chambers | StormTech LLC | www.stormtech.com |
| StormTrap® | StormTrap | www.stormtrap.com |
| Triton Chambers™ | Triton Stormwater Solutions | www.tritonsws.com |

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# VEGETATED FILTER STRIP

Structural Best Management Practice

Volume Control ☐

Quality Control ☑

*http://www.wsdot.wa.gov/Environment/WaterQuality/Research/Reports.htm*

| PERFORMANCE | | | |
|---|---|---|---|
| M | Sediment | L | Bacteria |
| M | Metals | H | Trash and debris |
| M | Oil and grease | L | Volume Reduction |
| L | Nutrients | L/M | Peak Flow Control |

H – High, M – Medium, L – Low
Note: Effectiveness levels are relative to other BMPs in this manual using typical designs. Design enhancements may change the designations.

*Applications*

- Road and highway shoulders
- Areas adjacent to small parking lots and driveways
- Residential, commercial or institutional landscaping

*Advantages*

✓ Recommended choice for a pre-treatment BMP
✓ Simple, aesthetically pleasing landscaping
✓ Low cost/ low maintenance

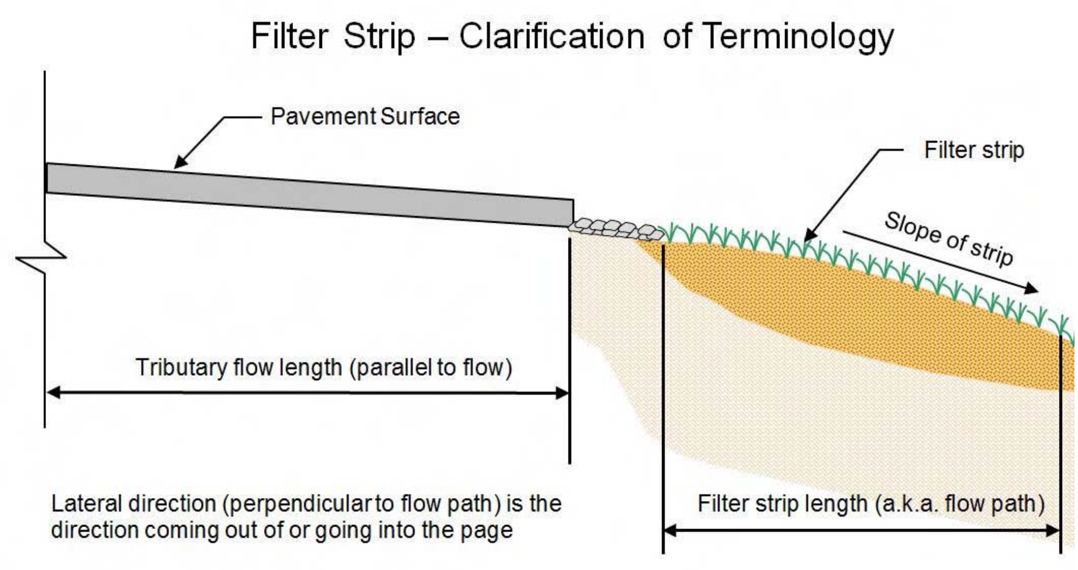
*Limitations*

- Must be sited adjacent to impervious surfaces
- May not be suitable for industrial land uses
- Requires sheet flow across vegetated area

VEGETATED FILTER STRIP

## DESCRIPTION

Vegetated filter strips (filter strips) are vegetated areas designed to treat sheet flow runoff from adjacent impervious surfaces or intensive landscaped areas such as golf courses. Filter strips decrease runoff velocity, filter out total suspended solids and associated pollutants, and provide some infiltration into underlying soils. While some assimilation of dissolved constituents may occur as runoff flows through the filter strip, these BMPs are generally more effective in trapping sediment and particulate-bound metals, nutrients, and pesticides. Filter strips are well suited to treat runoff from roads and highways, driveways, roof downspouts, small parking lots, and other impervious surfaces. They are also good for use as vegetated buffers between developed areas and natural drainages. **These BMPs filter storm water immediately adjacent to impervious surfaces and are typically intended for pre-treatment and not as a standalone BMP**. Filter strips are more effective when the runoff passes through the vegetation and thatch layer in the form of shallow, uniform "sheet flow".


Filter Strip – Clarification of Terminology
Pavement Surface
Filter strip
Slope of strip
Tributary flow length (parallel to flow)
Lateral direction (perpendicular to flow path) is the direction coming out of or going into the page
Filter strip length (a.k.a. flow path)

## SITE SUITABILITY CONSIDERATIONS

Filter strips can be used to treat a number of different surfaces, including highways, roads, driveways, parking lots, and landscaped areas. Other site suitability considerations are listed below:

| SITE SUITABILITY CONSIDERATIONS FOR VEGETATED FILTER STRIPS | |
|---|---|
| Tributary Area | < 2 acres; 87,000 $ft^2$ |
| Typical BMP area as percentage of tributary area (%) | >5 %[1] |
| Site slope (%) | 2-6% in flow direction; 4% maximum slope in lateral direction [2] |
| Depth to seasonally high groundwater | > 2 ft |
| Hydrologic soil group | Any[3] |

1 – Tributary area is the area of the site draining to the BMP. The maximum length of tributary area in the direction of flow is 150'. Tributary areas can be larger or smaller in some instances.
2 – If site slope exceeds that specified or is within 200 ft from the top of a hazardous slope or landslide area, a geotechnical investigation is required.
3 – Filter strips cannot be applied in areas with highly erodible soils.

The effectiveness of a filter strip is a function of the contributing land use, the size of the drainage area (i.e. width of tributary area perpendicular to flow), the slope, drainage area imperviousness, vegetation type, and the filter strip length in the direction of flow. Filter strips work most effectively when grading or natural topography allows for sheet flow from adjacent tributary areas. The topography of a site should allow for the design of a filter strip with sufficiently mild slope and flow capacity. Other site suitability issues are included below:

- Placement – Filter strips must be appropriately sited to avoid concentrated, erosive flows from entering the strip. Filter strips must be sited directly adjacent to their tributary land use, and the maximum length of the tributary area in the direction of flow towards the filter strip should be 150 feet.
- Minimum width of vegetated filter strip in flow direction – The minimum width in the direction of flow is 15 feet (25 preferred). This width requirement is related to the residence time needed to adequately remove sediment and pollutants as runoff flows through the strip.
- Development density – Filter strips may be challenging to implement in dense urban areas due to width (in the direction of flow) requirements needed for adequate residence times. However, filter strips can be used in roadway rights-of-way that may be planned and under-utilized for storm water mitigation.
- Adjacent Land Uses – Filter strips can be used to pre-treat runoff from impervious land uses, including roadways. However, filter strips should not be used near industrial sites or locations where spills may occur without a downstream water quality BMP designed specifically for such a scenario.
- Shade – Filter strips should be located away from buildings or dense tree canopies as excessive shade may lead to poor plant growth.

## DESIGN CRITERIA

Filter strips can be designed as a standalone water quality BMP or as pretreatment to another BMP. Filter strips are most commonly used for pretreatment for vegetated swales that have lateral inflow, but they can be used for pretreatment for virtually any BMP type provided there is a collection and conveyance system at the toe of the filter strip slope. The following table summarizes the minimum design criteria for filter strips. Additional sizing criteria and design guidance is provided in the subsections below.

| DESIGN PARAMETER | UNIT | DESIGN CRITERIA |
|---|---|---|
| Flood control design flow rate, $Q_{fc}$ | cfs | See SD1's Storm Water Rules and Regulations or Boone County's Subdivision Regulations for calculating $Q_{fc}$ |
| Water quality design flow rate, $Q_{wq}$ | cfs | See Chapter 3 for calculating $Q_{wq}$ |
| Maximum design flow depth | in | 1 |
| Design residence time | min | 5 |
| Maximum length of tributary area (parallel to flow) | ft | 150 |
| Minimum length in flow direction | ft | 15 (25 preferred); if sized for pretreatment only, filter strip can be a minimum of 5 ft |
| Maximum length in flow direction | ft | 150 |
| Slope of strip | % | 2-6 |
| Maximum lateral slope | % | 4 |
| Vegetation | - | Turf grass or approved equal (see Vegetation section below and Appendix C) |
| Vegetation height | in | 2-4 (typical) |
| Elevation of flow spreader | in | > 1 inch below pavement surface |

### *Geometry and Size*

- The width of the filter strip shall extend across the full width of the tributary area. The upstream boundary of the filter strip shall be located contiguous to the tributary area.
- The length of the filter strip (in direction of flow) shall be between 15 and 150 feet. A minimum length of 25 feet is preferred if used as a standalone BMP. Filter strips used for pretreatment shall be at least 5 feet long (in direction of flow).
- Filter strips shall be designed on slopes (parallel to the direction of flow) between 2% and 6%; steeper slopes tend to result in concentrated flow. Slopes less than 2% could pond runoff, and in low permeable soils, create a mosquito breeding habitat.
- The lateral slope of strip (parallel to the edge of the tributary area, perpendicular to the direction of flow) shall be 4% or less.
- Grading shall be even: a filter strip with uneven grading perpendicular to the flow path will develop flow channels over time.
- The top of the filter strip shall be installed at least 1 inch below the adjacent land surface to allow for vegetation and sediment accumulation at the edge of the strip. A beveled transition is acceptable and may be required per roadside design specifications. A flow spreader shall be installed between the filter strip and the land surface; see energy dissipation below.

- Both the top and toe of the slope shall be as flat as possible to encourage sheet flow and prevent channeling and erosion at these locations.

### Energy Dissipation/ Level Spreading

Runoff entering a filter strip must not be concentrated. A flow spreader shall be installed at the edge of the pavement to uniformly distribute the flow along the entire width of the filter strip.

- At a minimum, a gravel flow spreader (gravel-filled trench) shall be placed between the impervious area contributing flows and the filter strip, and meet the following requirements:
  - The gravel flow spreader shall be a minimum of 6 inches deep and should be 12 inches wide.
  - Where the ground surface is not level, the bottom of the gravel trench and the outlet lip should both still be constructed level.
  - Along roadways, gravel flow spreaders should be placed and designed in accordance with the appropriate local jurisdiction's road design specifications for compacted road shoulders.
- Curb ports and interrupted curbs (curb cuts) may only be used in conjunction with a gravel spreader to better ensure that water sheet flows onto the strip, provided:
  - Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to admit water to the filter strip. Interrupted curbs are sections of curb installed with gaps spaced at regular intervals along the total width of the treatment area.
  - Gaps should be at least every 6 feet to allow distribution of flows into the treatment facility before they become too concentrated. The opening should be a minimum of 11 inches. Approximately 15 percent or more of the curb section length should be in open ports, and as a general rule, no opening should discharge more than 10 percent of the overall flow entering the facility.
- Energy dissipaters are needed in filter strips if sudden slope drops occur, such as locations where flows in a filter strip pass over a rockery or retaining wall aligned perpendicular to the direction of flow. Adequate energy dissipation at the base of a drop section should be provided by a riprap pad.

### Water Depth, Velocity, and Residence Time

- For the water quality design flow rate, $Q_{wq}$, the design residence time (the time that it takes for water to flow across the filter strip) must be 5 minutes or greater for adequate treatment. The requirement can be waived if the filter strip is only used for pretreatment.
- To reduce the potential for erosion and the formation of rills across the filter strip surface, the flow velocity shall never exceed 4 ft/s.

### Soils

- Filter strip soils shall be amended with 2 inches of well-rotted compost, unless the organic content of the native soil is already greater than 10%. The compost shall be mixed into the native soils to a depth of 6 inches to prevent soil layering and washout of compost. The compost will contain no sawdust, green or under-composted material, or any other toxic or harmful substance. It shall contain no un-sterilized manure which can lead to high levels of potentially pathogenic bacteria in the runoff. In lieu of amending the native soils, the top 6 inches may be replaced with a bioretention soil mix. See Appendix B for bioretention mix design guidance.

### *Vegetation*

Filter strips must be uniformly graded and densely vegetated with erosion-resistant grasses that effectively bind the soil. Native or adapted grasses are preferred because they generally require less fertilizer and are more drought resistant than exotic plants. The following vegetation guidelines shall be followed for filter strips:

- Sod (turf) can be used instead of grass seed, as long as there is complete coverage.
- Irrigation shall be provided to establish the grasses.
- Grasses or turf shall be maintained at a height of 2 to 4 inches. Regular mowing is often required to maintain the turf grass cover.
- Trees or shrubs shall not be used in abundance because they shade the turf and impede sheet flow.
- See Appendix C for more information on recommended vegetation. The plant list in Appendix C shall be used as a guide only and shall not replace project-specific planting recommendations provided by a landscape professional including recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth.
- Grass and seed mixes shall be applied at a minimum rate of 0.5 lbs/1000 sq.ft.
- Prohibited non-native plant species will not be permitted. For information on invasive plant species in Kentucky, go to the Early Detection & Distribution Mapping System at http://www.eddmaps.org/tools/stateplants.cfm?id=us_ky

## DESIGN PROCEDURE

The flow capacity of a filter strip is a function of the longitudinal slope (parallel to flow), the resistance to flow (e.g., Manning's roughness), and the flow depth along the strip. If the filter strip is used for primary treatment, the dimensions must be adjusted such that the residence time is at least 5 minutes (see Steps 1 through 5). If the filter strip is used for pretreatment, it must be at least 5 feet wide and meet the maximum flow depth and velocities for the flood control design flow rate (Steps 5 and 6).

VEGETATED FILTER STRIP

### *Step 1: Calculate the Water Quality Design Flow*

The water quality design flow rate, $Q_{wq}$, shall be determined using the procedure provided in Chapter 3.

### *Step 2: Calculate the Minimum Filter Strip Width (Perpendicular to Flow)*

Determine the minimum width ($W_{min}$), perpendicular to flow, allowable for the filter strip to accommodate the design flow, and design for that width or larger.

$$W_{\min} = \frac{Q_{wq}}{q_{a,\min}}$$

Where:

$W_{min}$ = minimum width of filter strip (and tributary area) (ft)

$Q_{wq}$ = water quality design flow (cfs)

$q_{a,min}$ = maximum linear unit application rate, 0.005 cfs/ft

### *Step 3: Calculate the Design Flow Depth for Water Quality*

The water quality design flow depth ($d_{wq}$) can be calculated using a modified Manning's equation based on the dimensions of the filter strip width and slope:

$$d_{wq} = 12\left[\frac{n \cdot Q_{wq}}{1.49 \cdot W \cdot s^{0.5}}\right]^{0.6}$$

Where:

$d_{wq}$ = design flow depth (in)

$Q_{wq}$ = water quality design flow (cfs)

$n$ = Manning's roughness coefficient for shallow flow conditions (unitless); use 0.25.

$W$ = width of strip (and tributary area) (ft) (should be equal to or greater than $W_{min}$)

$s$ = longitudinal slope in flow direction (ft/ft) (should be within 0.02 - 0.06)

### *Step 4: Calculate the Filter Strip Design Velocity for Water Quality*

The design flow velocity ($V_{wq}$) is based on the design flow, design flow depth, and width of the strip and can be calculated using the following equation:

$$V_{wq} = \frac{Q_{wq}}{W\left(d_{wq}/12\right)}$$

Where:

$V_{wq}$ = water quality design velocity (ft/s)

$Q_{wq}$ = water quality design flow (cfs)

$d_{wq}$ = water quality design flow depth (in)

$W$ = width of strip (and tributary area) (ft) (should be equal to or greater than $W_{min}$)

### *Step 5: Calculate Filter Strip Length*

Calculate the filter strip length required to achieve the required minimum residence time using the following equation:

$$L = 60 \cdot t \cdot V_{wq}$$

Where:

$L$ = filter strip length (ft) (must be 15 ft to 150 ft for biotreatment)

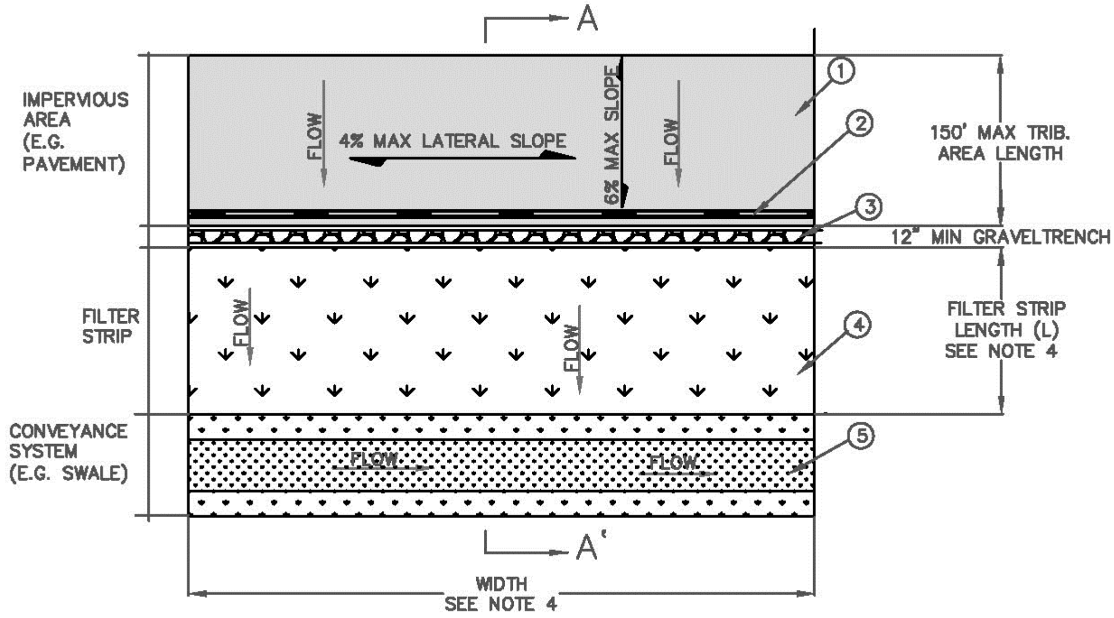
$t$ = hydraulic residence time (min) (minimum 5 minutes for biotreatment)

$V_{wq}$ = design flow velocity (ft/s)

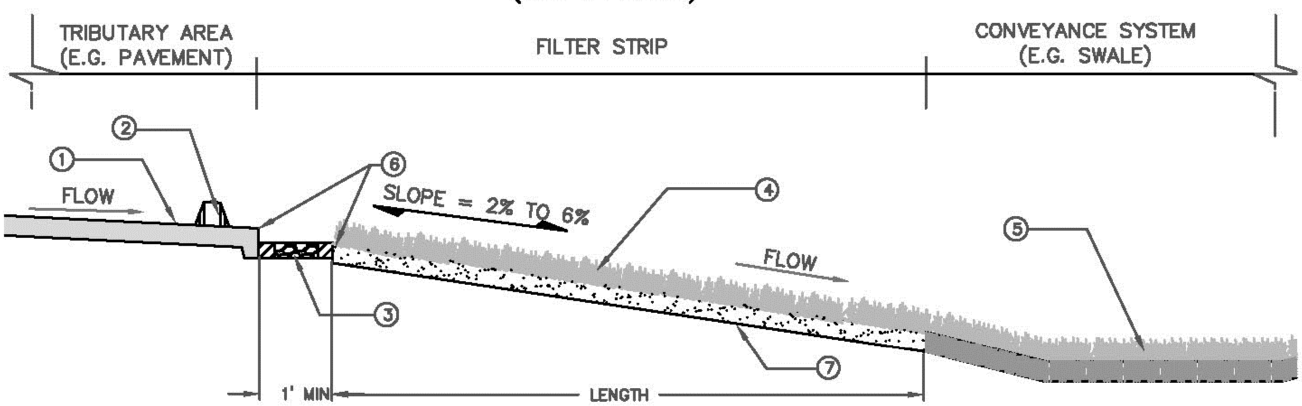
If the length calculated is less than 15 feet, design the filter strip with a length of at least 15 feet unless the strip is to be used for pretreatment only. If hydraulic residence time is less than five minutes, then increase the filter strip width or decrease the slope and return to Step 3.

## DESIGN SCHEMATICS

The following schematics should be used as further guidance for design of filter strips. Other designs are permissible if minimum design criteria are met.




Section A – A'
(Not to Scale)




NOTES:

1. MAXIMUM LENGTH OF IMPERVIOUS TRIBUTARY AREA (PER UNIT WIDTH OF FILTER STRIP) SHALL BE 150'.
2. OPTIONAL NOTCHED CURB SPREADERS MAY BE USED IN CONJUNCTION WITH GRAVEL FLOW SPREADER.
3. GRAVEL FLOW SPREADER 6" DEEP BY 12" WIDE MIN SHALL BE PROVIDED.
4. VEGETATED FILTER STRIP SURFACE SLOPE SHALL BE BETWEEN 2–6%. LENGTH OF STRIP MUST BE BETWEEN 15' AND 150'. WIDTH MUST BE EQUAL OR GREATER THAN THE WIDTH OF THE TRIBUTARY AREA.
5. INSTALL SWALE OR OTHER CONVEYANCE SYSTEM DOWNSTREAM OF FILTER STRIP.
6. TOP OF FILTER STRIP SHALL BE >1" BELOW TOP OF ADJACENT PAVEMENT.
7. AMEND SOILS WITH 2" OF COMPOST TILLED INTO 6" OF NATIVE SOIL UNLESS NATIVE SOIL ORGANIC CONTENT > 10%.

VEGETATED FILTER STRIP

## MAINTENANCE

Maintenance access shall be provided at the upper edge of a filter strip to enable maintenance of the inflow spreader throughout the strip width and allow access for mowing equipment.

| SCHEDULE | ACTIVITY |
|---|---|
| As needed (frequently) | • Mow vegetation to maintain design height of 2-4 inches.<br>• Maintain health of plants and remove any noxious weeds or plants that interfere with the function of the filter strip.<br>• Remove any trash and debris that has accumulated at the edge of the filter strip.<br>• Remove accumulation of fine sediment, dead leaves, etc. greater than 2 inches in depth or that covers the vegetation. |
| As needed (within 48 hours after every storm greater than 1 inch) | • Inspect filter strip for sediment accumulation.<br>• Inspect filter strip for erosion and/or scouring.<br>• Inspect flow spreader for uneven gravel depth or clogs. |
| As needed (infrequently) | • Repair any structural damage to flow spreader, level gravel, and remove/ repair clogs.<br>• Re-grade and re-vegetate to repair damage from major erosion (bare spots wider than 12 inches) if needed. |

## ADDITIONAL SOURCES OF INFORMATION


AMEC Earth and Environmental Center for Watershed Protection et al. Georgia Stormwater Management Manual. 2001.

Barrett, E.M., Walsh, M.P., and Malina Jr., J.F., 1998. Performance of Vegetative Controls for Treating Highway Runoff. *Journal of Environmental Engineering*. Volume 124, Issue 11, pp. 1121-1128.

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Deletic, A., and Fletcher, D.T., 2006. Performance of grass filters used for stormwater treatment—a field and modeling study. *Journal of Hydrology*, Volume 317, Issues 3-4, Pages 261-275.

Gharabaghi, B., Rudra, P.R., Whiteley, R.H., and Dickinson, T.W. Performance Testing of Vegetative Filter Strips. World Water and Environmental Resources Congress 2001 Orlando, Florida, USA,

Otto, S., Vianello, M., Infantino, A., Zanin, G., and Di Guardo, A., 2008. Effect of a Full-grown Vegetative Filter Strip on Herbicide Runoff: Maintaining of Filter Capacity Over Time. *Chemosphere*, Volume 71, Issue 1, Pages 74-82.

Pätzold, S., Klein, C., and Brümmer, G.W., 2007. Run-off Transport of Herbicides during Natural and Simulated Rainfall and its Reduction by Vegetated Filter Strips. *Soil Use and Management*, Volume 23, Number 3, pp. 294-305(12).

Sanitation District No. 1. Northern Kentucky Regional Storm Water Management Program: Rules and Regulations. 2011. Available at http://www.sd1.org/Resources.aspx?cid=9

Strecker, Eric and Klaus Rathfelder. Memo to Kentucky Sanitation District No. 1, Fort Wright, KY, 17 Nov. 2008.

Virginia Department of Conservation and Recreation. Virginia DCR Stormwater Design Specification No. WQ-1: Grass Filter Strips. 2011.

Washington Department of Transportation Vegetated Filter Strip Image: http://www.wsdot.wa.gov/Environment/WaterQuality/Research/Reports.htm

# CHAPTER 8
## EXAMPLE DESIGN CALCULATIONS

The following sections provide example design calculations for hypothetical development scenarios using the following BMPs:

- Bioretention;
- Biofiltration Swale;
- Extended Detention Basin; and
- Permeable Pavement.

## 8.1 EXAMPLE 1: BIORETENTION

### HYPOTHETICAL SITE DATA

A local developer plans to develop a small 10 acre subdivision located in a separate sewer area. The proposed impervious area is 4.0 acres after development, or 40.0 percent of the total area. The underlying soils on the site are hydrologic soil group C. The developer is considering using a bioretention facility to help capture the water quality design volume. An in-situ percolation test, performed at the elevation of the proposed bottom of the bioretention facility, indicated the native soil infiltration rate is 0.05 inches/hour.

### STEP 1: DETERMINE SITE SUITABILITY FOR BIORETENTION SYSTEMS

Per the Bioretention/Rain Garden Fact Sheet this site is suitable for bioretention BMPs. Generally, a tributary area to a bioretention system should be less than 5 acres; and therefore, this developer plans to install two bioretention areas in the subdivision, each with approximately 5 acres of tributary drainage area. Since the underlying soils have an infiltration rate less than 2 inches/hour, the bioretention system will be designed with an underdrain.

### STEP 2: COMPUTE WATER QUALITY DESIGN VOLUME

#### *Compute Volumetric Runoff Coefficient:*

$$R_v = 0.009 \cdot \%Imp + 0.05$$

Where:
$R_v$ = the volumetric runoff coefficient (unit less)
*% Imp* = the percent imperviousness of the site = 40.0%

$$\boldsymbol{R_v = 0.009 \cdot 40 + 0.05 = 0.41}$$

#### *Compute Water Quality Design Volume:*

Using the design storm volume, the water quality design volume may be computed using a modified form of the rational formula:

$$V_{wq} = 3630 \cdot R_v \cdot P \cdot A$$

Where:
$V_{wq}$ = the water quality design volume ($ft^3$)
$R_v$ = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage
$P$ = the rainfall depth of the storm (in) = 0.8 in (for new development)
$A$ = the BMP drainage area (acres)

$$\boldsymbol{V_{wq} = 3630 \cdot 0.41 \cdot 0.8\ in\ \cdot 5\ acres = 5{,}953\ ft^3}$$

## STEP 3: DETERMINE DESIGN INFILTRATION RATE

Since the facility includes the use of an underdrain, the design infiltration rate is based on that of the bioretention planting matrix. Per the planting matrix specified in on the Bioretention/Rain Gardens Fact Sheet, a design infiltration rate of 2 in/hr should be assumed and native soil infiltration rate multiplied by a factor of safety of 0.25.

$$k_{media} = 2\ in/hr$$

$$k_{native} = (0.25)(k_{measured})$$

Where:

$k_{measured}$ = the infiltration rate determined from in-situ test

$$k_{native} = (0.25)(k_{measured})$$

$$\boldsymbol{k_{native} = (0.25)(0.05\ in/hour) = 0.0125\ in/hour}$$

## STEP 4: COMPUTE FACILITY SURFACE AREA

The required surface area can be calculated using the following equation:

$$A = \frac{12V_{wq}}{d_p + \eta \cdot d_{media}}$$

Where:

$A$ = required area of bioretention area (ft$^2$)

$V_{wq}$ = water quality design volume (ft$^3$)

$d_p$ = design depth of ponding above bioretention area (12 inches or less)

$\eta =$ drainable porosity of the media (unit less); use 0.25 (This value is applicable to the bioretention soil mix specified in Appendix B. A different drainable porosity value may be approved with adequate documentation).

$d_m$ = depth of planting media (in) (minimum of 24 inches)

$$\boldsymbol{A = \frac{12 \cdot 5,953\ ft^3}{8 + 0.25 \cdot 24} = 5,103\ ft^2}$$

## STEP 5: COMPUTE FLOW CAPACITY OF UNDERDRAIN

Underdrains must be designed so they drain water from the rock layer substantially faster than water enters from the media layer above. The design flow capacity of the underdrain pipe can be computed as:

$$Q_{und} = f_s \frac{k_{media}(A)}{(12)(3600)}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$f_s$ = factor of safety (use 5)

$k_{media}$ = design infiltration rate (use 2 in/hr)

$A$ = area of bioretention (ft²)

$$\boldsymbol{Q_{und} = 5 \cdot \frac{2\ in/hr \cdot 5,103\ ft^2}{12 \cdot 3,600} = 1.18}\ \text{cfs}$$

## STEP 6: DETERMINE DIAMETER/NUMBER OF UNDERDRAIN PIPES

The diameter of a single pipe to convey the underdrain flow can be computed as:

$$D_s = 16 \cdot \left(\frac{(Q)(n)}{s^{0.5}}\right)^{3/8}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$D_s$ = single pipe diameter (in)

$n$ = Manning's roughness (use 0.011 for smooth pipe and 0.016 for corrugated pipe)

$s$ = pipe slope (recommended to be 0.005)

$$\boldsymbol{D_s = 16 \cdot \left(\frac{(1.18\ ft^3/s) \cdot (0.011)}{0.005^{0.5}}\right)^{3/8} = 8.47\ in}$$

$$\boldsymbol{\rightarrow\ use\ (1)\ 9\ in\ underdrain\ or\ try\ (2)\ underdrains}$$

If more than one pipe is used, then this formula should be used to determine the sizing of the combination of pipes so that the sum of the flow rates of each pipe used is greater than or equal to $Q_{und}$.

$$\boldsymbol{Q_{und2Pipes}} = \frac{Q_{und}}{2} \qquad \boldsymbol{Q_{und2Pipes}} = \frac{1.18}{2} = 0.59\ cfs$$

$$\boldsymbol{D_s = 16 \cdot \left(\frac{(0.59\ ft^3/s) \cdot (0.011)}{0.005^{0.5}}\right)^{3/8} = 6.5\ in}$$

$$\boldsymbol{\rightarrow\ try\ (3)\ underdrains}$$

$$\boldsymbol{Q_{und3Pipes}} = \frac{Q_{und}}{3} \qquad \boldsymbol{Q_{und3Pipes}} = \frac{1.18}{3} = 0.39\ cfs$$

$$\boldsymbol{D_s = 16 \cdot \left(\frac{(0.39\ ft^3/s) \cdot (0.011)}{0.005^{0.5}}\right)^{3/8} = 5.6\ in}$$

$$\boldsymbol{\rightarrow\ use\ (3)\ 6\ in\ underdrains}$$

## STEP 7: CHECK FACILITY DRAWDOWN TIME ABOVE UNDERDRAIN

Compute the drawdown of the facility above the underdrain. The drawdown computed here represents the drawdown of the ponded area plus the drawdown of the media storage area. It does not include the drawdown of the gravel layer. Compute the drawdown using the following equation to ensure that complete drawdown occurs in no more than 48 hours:

$$T_{Tot} = \left[ \frac{d_p + \eta_{media} d_{media}}{k_{media}} \right]$$

Where:

$T_{Tot}$ = total time to draw down both the ponded volume and the media volume (hours)

$d_p$ = design ponding depth (in) [max 8 inches]

$d_{media}$ = depth of planting media (in) [min 24 inches]

$k_{media}$ = media bed infiltration rate (in/hr); use 2 in/hr

$\eta_{media}$ = drainable porosity of the bioretention soil mix (unit less); use 0.25 (This value is applicable to the bioretention soil mix specified in Appendix B. A different drainable porosity value may be approved with adequate documentation).

$$\boldsymbol{T_{Tot} = \frac{8\mathrm{in} + 0.25 \cdot 24\mathrm{in}}{2in/hr} = 7\ hrs}$$

## BIORETENTION SYSTEM DESIGN

This local developer will install ***two*** bioretention systems with underdrains and each will meet the following design criteria:

- Facility drainage area: 5 acres; 40 percent impervious area
- Facility surface area: 5,103 $ft^2$
- Underdrain: (1) 9 in or (3) 6 in smooth pipes to be installed the entire length of the bioretention system
- Design ponding depth: 8 in
- Depth of planting media: 24 in
- Media bed infiltration rate (amended soil): 2 in/hr

## 8.2 EXAMPLE 2: BIOFILTRATION SWALE

### HYPOTHETICAL SITE DATA

A local developer plans to redevelop a 1 acre commercial site located in the separate sewer area. The proposed impervious area after redevelopment is 0.8 acre, or 80 percent of the total area. The underlying soils on the site are hydrologic soil group C. The developer is considering using a biofiltration swale to meet water quality design flow rates. An in-situ percolation test, performed at the elevation of the proposed bottom of the facility, indicated the native soil infiltration rate is 0.05 inches/hour. The proposed location for the biofiltration swale has a flow direction slope of 2 percent. This biofiltration is an off-line facility; and therefore, the flood control design flow rate is not included in the calculations.

### STEP 1: DETERMINE SITE SUITABILITY FOR BIOFILTRATION SWALE

Per the Biofiltration Swale Fact Sheet this site is suitable for a biofiltration swale. However, due to the condition of the in-situ soils, an underdrain should be incorporated in the design.

### STEP 2: COMPUTE WATER QUALITY DESIGN FLOW RATES

The water quality design flow rate, $Q_{wq}$, shall be determined using continuous runoff modeling techniques illustrated below. If the swale is on-line, the flood control design flow rate, $Q_{fc}$, must also be determined using the procedure provided in SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations.

#### *Compute Volumetric Runoff Coefficient:*

$$R_v = 0.009 \cdot \%Imp + 0.05$$

Where:
$R_v$ = the volumetric runoff coefficient (unit less)
$\%\ Imp$ = the percent imperviousness of the site = 80.0%

$$\boldsymbol{R_v = 0.009 \cdot 80 + 0.05 = 0.77}$$

#### *Compute Water Quality Design Flow Rate:*

The 80th percentile hourly rainfall intensity measured at the Cincinnati-Northern Kentucky Airport is approximately 0.08 in/hr (Strecker and Rathfelder, 2008). Therefore, doubling this intensity gives a **0.16 in/hr** design storm intensity, which can be converted to a design flow rate using the rational formula:

$$Q_{wq} = R_v \cdot i \cdot A$$

Where:
$Q_{wq}$ = the water quality design flow rate (cfs)
$R_v$ = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage
$i$ = rainfall intensity (in/hr) [use 0.16 in/hr]
A = the BMP drainage area (acres)

$$Q_{wq} = 0.77 \cdot 0.16\ in/hr \cdot 1\ acre = 0.123\ cfs$$

*Note that 1 acre-in/hr = 1.0083 cfs; this conversion factor can be used with Equation 7, but is not necessary as the uncertainty for the other parameters is generally well above 0.8%.*

## STEP 3: DETERMINE DEPTH AND RETENTION TIME REQUIREMENTS

Select a water quality design depth and retention time based on the permissible ranges for swales shown in the Design Criteria table included in the Biofiltration Swale Fact Sheet. It is recommended to start with a 2-inch (0.167 ft) water quality depth, $D_{wq}$, and a 9 minute water quality retention time, t. To achieve permissible values for the dimensions below, these initial values may need to be altered.

## STEP 4: COMPUTE SWALE BOTTOM WIDTH

Compute the bottom width of the swale using the following simplified form of the Manning's equation (side slopes neglected):

$$W = \frac{nQ_{wq}}{1.49\,(D_{wq}^{1.67})S^{0.5}}$$

| Determination of Manning's n |
|---|
| Medium grass: n = 0.15 |
| Dense grass: n = 0.25 |
| Very dense Bermuda-type grass: n = 0.35 |

Where:
W = channel bottom width (ft)
n = Manning's "n" (dimensionless)
$Q_{wq}$ = water quality design flow (cfs)
$D_{wq}$ = water quality flow depth (ft)
S = longitudinal slope (ft/ft)

$$W = \frac{0.25 \cdot (0.123\ ft^3/s)}{1.49\,((0.167ft)^{1.67}) \cdot ((0.02\ ft/ft)^{0.5})} = 2.90\,ft$$

Since the bottom width is less than 3 feet, set W = 3 feet and recalculate the water quality design flow depth ($D_{wq}$):

$$D_{wq} = \left[\frac{nQ_{wq}}{1.49WS^{0.5}}\right]^{0.6}$$

Where:
*W* = channel bottom width (ft); use 3 feet
*n* = Manning's roughness coefficient for shallow flow conditions (unit less); use 0.25.
$Q_{wq}$ = water quality design flow (cfs)
$D_{wq}$ = water quality flow depth (ft)
*S* = longitudinal slope (ft/ft)

$$D_{wq} = \left[\frac{0.25 \cdot 0.123\, ft^3/s}{1.49 \cdot (3ft) \cdot ((0.02\, ft/ft)^{0.5})}\right]^{0.6} = 0.163\, ft$$

If bottom width was between 3 and 7 feet, $D_{wq}$ would not need to be recalculated and the designer would proceed to the next step, but if the calculated bottom width is more than 7 feet the designer would need to increase longitudinal slope (s), increase design flow depth ($D_{wq}$) to a maximum of 0.33 ft (4 in), install flow divider and flow spreader, or relocate swale downstream of a detention facility.

## STEP 5: CHECK FLOW VELOCITY

Compute the water quality design velocity, $V_{wq}$, using the bottom width and neglecting side slopes:

$$V_{wq} = \frac{Q_{wq}}{WD_{wq}}$$

Where:
$V_{wq}$ = water quality design flow (cfs)
$Q_{wq}$ = water quality design flow (cfs)
$W$ = channel bottom width (ft); use 3 feet
$D_{wq}$ = water quality flow depth (ft)

$$V_{wq} = \frac{0.123\, ft^3/s}{(3\, ft) \cdot (0.163\, ft)} = 0.25\, ft/s$$

Since $V_{wq}$ is less than 1 ft/s proceed to Step 6. However, if it was calculated to be greater than 1 ft/s, the designer needs to go back to Step 4 and modify longitudinal slope, bottom width (need flow divider if >7feet), or depth. $V_{wq}$ is less than 1 ft/s proceed to Step 6.

## STEP 6: COMPUTE THE REQUIRED SWALE LENGTH

Compute the minimum length of the swale:

$$L = tV_{wq}$$

Where:
$V_{wq}$ = water quality design flow (cfs)
t= residence time (seconds); 5 minutes (300 sec) minimum; >9 minutes (540 sec) preferred.

$$L = (540\, sec) \cdot (0.25\, ft/s) \ = 135 \ \ ft$$

## STEP 7: CHECK FLOOD CONTROL CONVEYANCE REQUIREMENTS (IF ON-LINE)

If the swale is an online storm water conveyance feature it shall be sized to provide conveyance for the flood control design flow rate $Q_{fc}$, with at least six inches of freeboard per SD1's Storm Water Rules and Regulations and Boone County Subdivision Regulations. Since this example is an off-line facility $Q_{fc}$ does not need to be checked.

## BIOFILTRATION SWALE DESIGN

This local developer will install a biofiltration swale that will meet the following design criteria:

- Facility drainage area: 1 acres; 80 percent impervious area
- Longitudinal slope in direction of flow: 2%
- Facility bottom width: 3 ft
- Facility length: 135 ft

## 8.3 EXAMPLE 3: EXTENDED DETENTION BASINS

### HYPOTHETICAL SITE DATA

A local developer plans to develop a 15 acre subdivision located in a separate sewer area. The proposed impervious area is 6.0 acres after development, or 40.0 percent of the total area. The proposed location of the extended detention basin is constrained to a total maximum width of 70 ft. The developer is considering using a extended detention basin to help capture the water quality design volume. An in-situ percolation test indicated the native soil infiltration rate is 0.05 inches/hour.

### STEP 1: DETERMINE SITE SUITABILITY FOR EXTENDED DETENTION BASINS

Per the extended detention basin fact sheet, extended detention basins should be sized to contain the total design volume plus 5% for sediment storage plus the freeboard requirements. Standard grading design should be implemented to estimate excavation and embankment fill quantities necessary while meeting the minimum design requirements described above. This site is suitable for a extended detention basin.

### STEP 2: COMPUTE WATER QUALITY DESIGN VOLUME

#### *Compute Volumetric Runoff Coefficient:*

$$R_v = 0.009 \cdot \%Imp + 0.05$$

Where:
$R_v$ = the volumetric runoff coefficient (unit less)
$\% \ Imp$ = the percent imperviousness of the site = 40.0%

$$\boldsymbol{R_v = 0.009 \cdot 40 + 0.05 = 0.41}$$

#### *Compute Water Quality Design Volume:*

Using the design storm volume, the water quality design volume may be computed using a modified form of the rational formula:

$$V_{wq} = 3630 \ \cdot R_v \cdot P \cdot A$$

Where:
$V_{wq}$ = the water quality design volume (acre-feet)
$R_v$ = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage
$P$ = the rainfall depth of the storm (in) = 0.8 in (for new development)
$A$ = the BMP drainage area (acres)

$$\boldsymbol{V_{wq} = 3630 \cdot 0.41 \cdot 0.8 \ in \cdot 15 \ acres = 17,860 \ ft^3}$$

## STEP 3: CALCULATE PRELIMINARY GEOMETRY BASED ON SITE CONSTRAINTS

Determine the active volume of the forebay using the fractional volume ($FV_{fb}$) requirements for the forebay (10-20%) plus 5% for sediment accumulation. Similarly determine active volume of main cell using the fractional volume ($FV_{mc}$) requirements for the main basin (80-90%).

$$V_{fb} = 1.05 V_{wq} \frac{FV_{fb}}{100}$$

$$V_{mc} = 1.05 V_{wq} \frac{FV_{mc}}{100}$$

Where:
$V_{wq}$ = total water quality volume of extended detention ($ft^3$)
$FV_{fb}$ = fractional water quality volume of forebay (10 to 20%)
$FV_{mc}$ = fractional water quality volume of main cell (80 to 90%)
$V_{fb}$ = volume of forebay ($ft^3$)

$$\boldsymbol{V_{fb} = 1.05 \cdot \left(17,860 ft^3\right) \frac{15}{100} = 2,813\ ft^3}$$

$$\boldsymbol{V_{mc} = 1.05 \cdot \left(17,860 ft^3\right) \frac{85}{100} = 15,940\ ft^3}$$

Calculate surface area of forebay and main cell using average depths.

$$A_{fb} = \frac{V_{fb}}{D_{fb}}$$

$$A_{mc} = \frac{V_{mc}}{D_{mc}}$$

Where:
$A_{fb}$ = Active forebay surface area ($ft^2$)
$A_{mc}$ = Active main cell surface area ($ft^2$)
$V_{fb}$ = volume of forebay ($ft^3$)
$V_{mc}$ = volume of main cell ($ft^3$)
$D_{fb}$ = average depth of forebay (ft)
$D_{mc}$ = average depth of main cell (ft)

$$\boldsymbol{A_{fb} = \frac{2,813\ ft^3}{1\ ft} = 2,813\ ft^2}$$

$$\boldsymbol{A_{mc} = \frac{15,940\ ft^3}{2.5\ ft} = 6,376\ ft^2}$$

Select either a width or length for the facility based on site constraints and the space available and calculate remaining dimensions using the surface areas for the forebay and the main cell.

> *Site constraints require this facility to have a minimum width of 70 ft, including the facility side slopes. Therefore, the minimum width of the forebay and main cell is 50 ft, which results in a forebay length of 56 ft and a main cell length of 128 ft.*

Calculate the non-active volumes and dimensions of the facility including berms, embankments and space needed for sediment storage. Add the non-active dimensions to the dimensions of the active forebay and main cell components to obtain the footprint dimensions of the facility.

### *Non-active dimensions:*

> *Per the extended detention basin fact sheet, the recommended side slopes is 4:1 for all slopes that will be mowed. With a 1 ft depth for the forebay, an additional 4 ft needs to added to the width and length of the forebay, and with 2.5 ft depth for the main cell, an additional 10 ft needs to be added to the width and length of the main cell.*

## STEP 4: SELECT FLOW CONTROL STRUCTURES AND CALCULATE OUTLET STRUCTURE DIMENSIONS

Provide adequate energy dissipation at inlets and size stilling basins as needed to prevent erosion. Recommended methods for sizing outlet structures for meeting the water quality drain time requirements and matching pre-development peak discharges are provided in Appendix E. Emergency spillways should be sized to convey the routed 100-yr design storm post-development peak flow rate. Refer to SD1's Storm Water Rules and Regulations or Boone County's Subdivision Regulation for acceptable methods for computing flood control design flows.

## EXTENDED DETENTION BASIN DESIGN

This local developer will install a extended detention basin that meets the following design criteria:

- Facility drainage area: 15 acres; 40 percent impervious area
- Forebay dimensions (including 4:1 side slopes):
  - Width: 50 ft + (2)8 = 66 ft
  - Length: 56 ft + (2)8 = 72 ft
  - Depth: 1 ft
- Main cell dimensions (including 4:1 side slopes):
  - Width: 50 ft + (2)10 = 70 ft
  - Length: 128 + (2)10 = 148 ft
  - Depth: 2.5 ft
- Outlet control structure to be designed per SD1's Storm Water Rules and Regulations or Boone County's Subdivision Regulation

## 8.4 EXAMPLE 4: PERMEABLE PAVEMENT

### HYPOTHETICAL SITE DATA

A local developer plans to redevelop a 1 acre commercial establishment located in a separate sewer area. The proposed impervious area is 0.7 acres, or 70.0 percent of the total area. The underlying soils on the site are hydrologic soil group B. An in-situ percolation test, performed at the elevation of the proposed bottom of the facility, indicated the native soil infiltration rate is 0.25 inches/hour.

### STEP 1: DETERMINE SITE SUITABILITY FOR PERMEABLE PAVEMENT

Per the Permeable Pavement Fact Sheet this site is suitable for permeable pavement systems with underdrains because the measured infiltration rate is less than 2.0 in/hr.

### STEP 2: COMPUTE WATER QUALITY DESIGN VOLUME

***Compute Volumetric Runoff Coefficient:***

$$R_v = 0.009 \cdot \%Imp + 0.05$$

Where:
$R_v$ = the volumetric runoff coefficient (unit less)
$\% \, Imp$ = the percent imperviousness of the site = 70.0%

$$\boldsymbol{R_v = 0.009 \cdot 70 + 0.05 = 0.68}$$

***Compute Water Quality Design Volume:***

Using the design storm volume, the water quality design volume may be computed using a modified form of the rational formula:

$$V_{wq} = 3630 \cdot R_v \cdot P \cdot A$$

Where:
$V_{wq}$ = the water quality design volume (acre-feet)
$R_v$ = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage
$P$ = the rainfall depth of the storm (in) = 0.4 in (for redevelopment)
$A$ = the BMP drainage area (acres)

$$\boldsymbol{V_{wq} = 3630 \cdot 0.68 \cdot 0.4\, in \cdot 1\, acre = 987.4\, ft^3}$$

### STEP 3: DETERMINE DESIGN INFILTRATION RATE

The design infiltration rate is based on the hydraulic conductivity of the native soil as determined using an in-situ percolation test measured at the elevation of the proposed bottom of the facility or at the depth of a limiting layer multiplied by a factor of safety of 0.25:

$$k_{native} = 0.25 \cdot k_{measured}$$

Where:
$k_{native}$ = the design infiltration rate for the native soils (in/hr)
$k_{measured}$ = the measured infiltration rate (in/hr)

$$\boldsymbol{k_{native} = 0.25 \cdot 0.25 = 0.0625\ \mathbf{in/hr}}$$

## STEP 4: DETERMINE THE 48-HOUR EFFECTIVE DEPTH

Determine the effective depth of water that can be drawn down within 48 hours.

$$d_{48} = \left(\frac{48}{12}\right) \cdot k_{design}$$

Where:
$d_{48}$ = effective depth of water that can be drawn down in 48 hours (ft)
$k_{design}$ = design infiltration rate determined in Step 3 (in/hr).

$$\boldsymbol{d_{48} = \frac{48}{12} \cdot \left(0.0625 \frac{\mathbf{in}}{\mathbf{hr}}\right) = 0.25\ ft}$$

## STEP 5: DETERMINE THE AGGREGATE RESERVOIR DEPTH

The depth of water stored in the gravel reservoir (everything below the invert of the underdrain, if one is present) should be equal or less than $d_{48}$. Determine the effective reservoir depth such that:

$$d_r \leq \frac{d_{48}}{\eta_r}$$

Where:
$d_{48}$ = effective depth of water that can be drawn down in 48 hours (ft)
$\eta_r$ = porosity of aggregate reservoir fill (unit less) [use 0.32 unless aggregate-specific data available]
$d_r$ = depth of gravel drainage layer below the invert of the underdrain, if present, or below the bedding course if no underdrain (ft)

$$\boldsymbol{d_r = \frac{0.25\ ft}{0.32} = 0.781\ ft}$$

## STEP 6: CALCULATE THE REQUIRED INFILTRATING AREA

The required infiltrating area can be calculated using the following equation:

$$A_{inf} \geq V_{wq} / (n_r \times d_r)$$

Where:

$A_{inf}$ = required infiltration area ($ft^2$)

$V_{wq}$ = water quality design volume ($ft^3$)

$n_r$ = porosity of aggregate reservoir fill (unit less) [use 0.32 unless aggregate-specific data available]

$d_r$ = depth of gravel drainage layer below the base of the underdrain, if present, or below the bedding course if no underdrain (ft)

$$\mathbf{A}_{inf} \geq \frac{987.4\, ft^3}{(0.32 \cdot 0.781\, ft)} = 3{,}950\, ft^2$$

If $A_{inf}$ is less than the planned permeable pavement area, the drainage area may be increased (repeat steps 2 and 6 to do this). If $A_{inf}$ is greater than the planned permeable pavement area, then the drainage area must be decreased. As a rule of thumb, the ratio of total tributary area (including the porous pavement) to the area of the porous pavement should not exceed 4:1 for porous asphalt or concrete and 2:1 for porous pavers. If there is no underdrain, larger drainage areas are permissible if the water quality design volume can be fully infiltrated and the tributary area yields low sediment loads. If there is an underdrain and the computed $d_r$ is less than 6 inches, the tributary area ratio does not need to be reduced below the maximum ratios listed above.

## STEP 7: FLOW CAPACITY OF UNDERDRAIN

Underdrains must be designed so they drain water from the rock layer quickly enough that the pavement above does not flood. The design flow capacity of the underdrain pipe can be computed as:

$$Q_{und} = f_s \frac{k_{media} \cdot A}{(12)(3600)}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$f_s$ = factor of safety [use 3]

$k_{media}$ = design infiltration rate (in/hr) [use 2 in/hr]

*A* = area of permeable pavement or infiltration area ($ft^2$)

$$Q_{und} = 3 \cdot \left( \frac{\left(2\, {}^{in}\!/_{hr}\right) \cdot 3{,}950\, ft^2}{(12)(3600)} \right) = 0.549\, cfs$$

## STEP 8: NUMBER OF UNDERDRAIN PIPES

The diameter of a single pipe to convey the underdrain flow can be computed as:

$$D_s = 16 \cdot \left( \frac{Q_{und} \cdot n}{s^{0.5}} \right)^{3/8}$$

Where:

$Q_{und}$ = required flow capacity of underdrain (cfs)

$D_s$ = single pipe diameter (in)

$n$ = Manning's roughness (use 0.011 for smooth pipe and 0.016 for corrugated pipe)

$s$ = pipe slope (recommended to be 0.005)

$$\boldsymbol{D_s = 16 \cdot \left(\frac{(0.549\ cfs) \cdot 0.011}{0.005^{0.5}}\right)^{3/8} = 6.36\ \mathrm{in}}$$

$$\rightarrow \boldsymbol{use\ (1) 8\ in\ underdrains\ or\ try\ (2) underdrains}$$

If more than one pipe is used, then this formula should be used to determine the sizing of the combination of pipes so that the sum of the flow rates of each pipe used is greater than or equal to $Q_{und}$.

$$\boldsymbol{Q_{und2Pipes}} = \frac{Q_{und}}{2} \qquad \boldsymbol{Q_{und2Pipes}} = \frac{0.549}{2} = 0.275\ cfs$$

$$\boldsymbol{D_s = 16 \cdot \left(\frac{(0.275\ ft^3/s) \cdot (0.011)}{0.005^{0.5}}\right)^{3/8} = 4.91\ in}$$

$$\rightarrow \boldsymbol{use\ (2)\ 6\ in\ underdrains}$$

## PERMEABLE CONCRETE DESIGN

This local developer will install permeable concrete that meets the following design criteria:

- Facility drainage area: 1 acres; 70 percent impervious area
- Facility surface area: 3,950 ft$^2$
- Underdrain: (1) 8 in or (2) 6 in smooth pipes to be installed the entire length of the system
- Depth of gravel layer below invert of underdrain pipe: 0.781 ft = 9.4 in

NOTE: Per the permeable pavement fact sheet: "*in areas where poor permeability preclude infiltration, the underdrain should be placed at the bottom of the reservoir layer and this layer may be decreased to one foot in thickness.*"

## BMP PERFORMANCE OF POTENTIAL TREATMENT BMP OPTIONS FOR REMOVAL OF THE PRIMARY POLLUTANTS OF CONCERN

BMP performance information, if available, can be used to verify the capabilities of potential treatment BMP options for removal of primary pollutants of concern. The following subsections provide summaries of BMP water quality performance for all of the BMPs presented in Tables 6.1-3 and 6.1-4 in Chapter 6 of the BMP Manual, except infiltration BMPs (i.e. vegetated swales, filter strips, bioretention, etc.), and green roofs. It is assumed that infiltration facilities discharge to the subsurface and will therefore remove 100 percent of the pollutants that enter the facilities as long as all siting conditions are met (e.g., depth to groundwater, infiltration rate, etc). Green roofs treat direct rainfall (i.e., they do not treat pollutants that are picked up from impervious surfaces); therefore, treatment effectiveness of green roofs are not comparable to other BMPs that treat runoff from a wide range of impervious surfaces that generally have higher pollutant concentrations. For similar reasons, subsurface BMPs (i.e. subsurface vaults) are not comparable to other BMPs since these BMPs mainly capture roof runoff. However, both of these systems are highly effective due to their ability to reduce runoff volumes as well as to capture pollutants that originate from the atmosphere during rainfall as well as from typical roofing materials that can dry-deposited on roofing surfaces.

The BMP performance data provided in the following subsections was compiled from a range of sources with the majority of performance data originating from the International Stormwater BMP Database (BMP Database), the most comprehensive source of BMP performance data (www.bmpdatabase.org). The first subsection below provides a summary of performance data from the BMP Database for the following BMP categories: detention basins (or dry ponds), retention basins (or wet ponds), wetland basins, biofilters, media filters, hydrodynamic devices, and porous pavement. Additional performance evaluations from research institutions and private testing facilities are provided in subsequent subsections for bioretention areas and proprietary devices.

Results from the International Stormwater BMP Database

The BMP Database includes over 340 BMP monitoring studies of a wide range of post-construction storm water treatment BMPs. It was developed in an effort to provide scientifically sound information to improve the selection, design, and performance of BMPs. Performance data in the BMP Database are analyzed annually to assist users in assessing the performance of different treatment BMP types for removal of pollutants of concern and for achieving hydromodification control. For the purposes of an overview analysis, BMP types are lumped into defined BMP categories and performance results are reported based on BMP category. These categories are: dry detention basin, wet pond, wetland basins, biofilters, media filters, hydrodynamic devices, and porous pavement. The biofilter category includes vegetated swales and filter strips. The media filter category includes bioretention areas, storm water planters, tree box filters, sand filters, and media cartridge filters.

Performance data in the BMP Database are based on a standardized reporting protocol that is required to be followed in order for data to be accepted into the database. In order for a BMP monitoring study to be considered for inclusion in the BMP Database, several criteria must be met, including:

(1) The study must be for a post-construction, permanent BMP conducted in the field. Laboratory studies are not accepted;

(2) Required fields in data entry spreadsheets must be provided, or explained if not applicable to the specific study. As a general rule, event mean concentrations (EMCs) are required for most studies, unless special considerations are identified (e.g., bacterial data may be taken as grab samples); and,

(3) Studies conducted by vendors or manufacturers of proprietary devices must meet certain requirements to ensure study results are independent and unbiased.

Specific information that is required to be collected and entered into the database includes general test site information, watershed information, general BMP information including cost data, monitoring stations, descriptions of each monitoring event including QA/QC protocols, monitoring results, and BMP design information. The monitoring results may include precipitation, storm runoff or base flow, water quality data, and/or settling velocity distributions associated with a monitoring event. The water quality parameters entered into the database are varied but most studies report the most common storm water pollutants including total suspended solids (TSS), phosphorus (total and dissolved), nitrogen (total, nitrate, and TKN), lead (total and dissolved), zinc (total and dissolved), and copper (total and dissolved). Bacteria (E. coli and/or fecal coliform) is also commonly measured although results vary widely even within a single BMP type. Organic compounds (e.g., hydrocarbons, pesticides, dioxins, oil & grease), often measured as specific parameters or total organic carbon (TOC), and oxygen demanding substances (e.g., sewage, food waste, green waste), measured as biological oxygen demand (BOD) or chemical oxygen demand (COD), are also collected in the database but are less common. The removal of trash and debris is oftentimes measured qualitatively or as a removal efficiency based on weight. It is not included as a parameter in the database.

Table A-1 summarizes the overview analysis results from available monitoring data drawn from the International Stormwater BMP Database as of October 2007. The analysis does not include results for bacteria, organic compounds, oxygen demanding substances, and volume reduction which will be described separately below. The data in Table A-1 can be used to determine whether any differences in treatment performance may be determined based on BMP category (e.g., detention basin, media filter, wetland, etc.). Summary statistics (in parentheses) are provided for the median and upper and lower 95th percentile confidence limits for the median of all influent and effluent event mean concentrations (EMCs) (one value per BMP study). A non-parametric statistical analysis of the difference in median values was performed to determine if there was a significant difference between median influent and effluent values. For each water quality constituent examined, only those BMP studies reporting at least three influent and effluent EMCs were included in the analysis data set. The database may contain additional studies not included in these analysis results due to unique site features or monitoring designs that may also be useful in assessing BMP performance.

It is important to note that the broad BMP categories may mask distinctive differences in design and performance in subcategories for multiple BMP types. This is particularly true for the Hydrodynamic Device (HD) category, which represents a wide range of various proprietary and non-proprietary device types. Each of the BMPs categorizes as HD device types incorporates or emphasizes a number of different methods of pollutant removal and design elements (e.g., storage versus flow-through designs, inclusion of media filtration, etc.) that vary significantly throughout the category. These design features likely have significant effects on BMP performance and the underlying detailed data analysis for each HD device should be referenced before drawing conclusions on the performance of HD devices (and to some extent other BMP types). At this time it is not possible to identify which methods of pollutant removal or design elements represent key differentiators in performance, or to further subdivide this category. Any interpretation or use of the results presented herein should fully acknowledge the widely varied nature of HD devices, as well as other BMP categories. It is recommended for HD devices in particular that more attention be paid to the observed ranges in performance than median or mean effluent values. Future plans for the BMP Database include developing additional BMP categories (and subcategories) as more studies become available.

Table A-1. Median of Average Influent and Effluent Concentrations of the BMPs from the International Stormwater BMP Database

| Constituent | Sample Location | Regional BMPs | | | Distributed BMPs | | | |
|---|---|---|---|---|---|---|---|---|
| | | Dry Detention Basin (n=25)[1] | Wet Pond (n=46)[1] | Wetland Basin (n=19)[1] | Biofilter (n=57)[1,2] | Media Filter (n=38)[1,3] | Hydrodynamic Devices (n=32)[1,4] | Porous Pavement (n=6)[1] |
| Total Suspended Solids (mg/L) | Influent | 72.65 (41.70-103.59) | 34.13 (19.16-49.10) | 37.76 (18.10-53.39) | 52.15 (41.41-62.88) | 43.27 (27.25-59.58) | 39.61 (21.95-76.27) | xx |
| | Effluent | 31.04 (16.07-46.01) | 13.37 (7.29-19.45) | 17.77 (9.26-26.29) | 23.92 (15.07-32.78) | 15.86 (9.74-21.98) | 37.67 (21.28-54.02) | 16.96 (5.90-48.72) |
| Total Phosphorus (mg/L) | Influent | 0.19 (0.17-0.22) | 0.21 (0.13-0.29) | 0.27 (0.11-0.43) | 0.25 (0.22-0.28) | 0.20 (0.15-0.26) | 0.24 (0.01-0.46) | xx |
| | Effluent | 0.19 (0.12-0.27) | 0.12 (0.09-0.16) | 0.14 (0.04-0.24) | 0.34 (0.26-0.41) | 0.14 (0.11-0.16) | 0.26 (0.12-0.48) | 0.09 (0.05-0.15) |
| Dissolved Phosphorus (mg/L) | Influent | 0.09 (0.06-0.13) | 0.09 (0.06-0.13) | 0.10 (0.04-0.22) | 0.09 (0.07-0.11) | 0.09 (0.03-0.14) | 0.06 (0.01-0.11) | xx |
| | Effluent | 0.12 (0.07-0.18) | 0.08 (0.04-0.11) | 0.17 (0.03-0.31) | 0.44 (0.21-0.67) | 0.09 (0.07-0.11) | 0.09 (0.04-0.13) | xx |
| Total Nitrogen (mg/L) | Influent | 1.25 (0.83-1.66) | 1.64 (1.39-1.94) | 2.12 (1.58-2.66) | 0.94 (0.94-1.69) | 1.31 (1.19-1.42) | 1.25 (0.33-2.16) | xx |
| | Effluent | 2.72 (1.81-3.63) | 1.43 (1.17-1.68) | 1.15 (0.82-1.62) | 0.78 (0.53-1.03) | 0.76 (0.62-0.89) | 2.01 (1.37-2.65) | xx |
| TKN (mg/L) | Influent | 1.45 (0.97-1.94) | 1.26 (1.03-1.49) | 1.15 (0.81-1.48) | 1.80 (1.62-1.99) | 1.52 (1.07-1.96) | 1.09 (0.52-1.67) | xx |
| | Effluent | 1.89 (1.58-2.19) | 1.09 (0.87-1.31) | 1.05 (0.82-1.34) | 1.51 (1.24-1.78) | 1.55 (1.22-1.83) | 1.48 (0.87-2.47) | 1.23 (0.44-3.44) |
| Nitrate-Nitrogen (mg/L) | Influent | 0.70 (0.35-1.05) | 0.36 (0.21-0.51) | 0.22 (0.01-0.47) | 0.59 (0.44-0.73) | 0.41 (0.30-0.51) | 0.40 (0.06-0.73) | xx |
| | Effluent | 0.58 (0.25-0.91) | 0.23 (0.13-0.37) | 0.13 (0.07-0.26) | 0.60 (0.41-0.79) | 0.82 (0.60-1.05) | 0.51 (0.08-1.34) | xx |

[1] Actual number of BMPs reporting a particular constituent may be greater or less than the number reported in this table, which was based on number of studies reported in the database based on BMP category.

[2] The biofilter BMP category includes vegetated swales and filter strips.

[3] The media filter BMP category includes sand filters, organic filters, gravel filters, bioretention filters, and proprietary filters.

[4] The hydrodynamic device BMP category includes a wide range of proprietary and non-proprietary hydrodynamic device types, catch basins, and oil/water separators.

Notes: xx - Lack of sufficient data to report median and confidence interval. Values in paranthesis are the 95% confidence intervals about the median. Differences in median influent and effluent concentrations does not necessarily indicated that there is a statistically significant difference between influent and effluent. See *Analysis of Treatment System Performance, International Stormwater BMP Database (1999-2007) (Geosyntec and WWE, 2007) for more detailed information.

*Source: International Stormwater Best Management Practices (BMP) Database June 2008 (www.bmpdatabase.org)*

Table A-1 (cont.). Median of Average Influent and Effluent Concentrations of the BMPs from the International Stormwater BMP Database

| Constituent | Sample Location | Regional BMPs | | | Distributed BMPs | | | |
|---|---|---|---|---|---|---|---|---|
| | | Dry Detention Basin (n=25)[1] | Wet Pond (n=46)[1] | Wetland Basin (n=19)[1] | Biofilter (n=57)[1,2] | Media Filter (n=38)[1,3] | Hydrodynamic Devices (n=32)[1,4] | Porous Pavement (n=6)[1] |
| Total Lead (µg/L) | Influent | 25.01 (12.06-37.95) | 14.36 (8.32-20.40) | 4.62 (1.43-11.89) | 19.53 (10.11-28.95) | 11.32 (6.09-16.55) | 18.12 (5.70-30.53) | xx |
| | Effluent | 15.77 (4.67-26.87) | 5.32 (1.63-9.01) | 3.26 (2.31-4.22) | 6.70 (2.81-10.59) | 3.76 (1.08-6.44) | 10.56 (4.27-16.85) | 7.88 (1.64-37.96) |
| Dissolved Lead (µg/L) | Influent | 1.25 (0.33-2.17) | 3.40 (1.12-5.68) | 0.50 (0.33-0.67) | 2.25 (0.77-3.74) | 1.44 (1.05-1.82) | 1.89 (0.83-2.95) | xx |
| | Effluent | 2.06 (0.93-3.19) | 2.48 (0.98-5.36) | 0.87 (0.85-0.89) | 1.96 (1.26-2.67) | 1.18 (0.77-1.60) | 3.34 (2.22-4.47) | xx |
| Total Zinc (µg/L) | Influent | 111.56 (51.50-171.63) | 60.75 (45.23-76.27) | 47.07 (24.47-90.51) | 176.71 (128.28-225.15) | 92.34 (52.29-132.40) | 119.08 (73.50-164.67) | xx |
| | Effluent | 60.20 (20.70-99.70) | 29.35 (21.13-37.56) | 30.71 (12.80-66.69) | 39.83 (28.01-51.56) | 37.63 (16.80-58.46) | 80.17 (52.72-107.61) | 16.60 (5.91-46.64) |
| Dissolved Zinc (µg/L) | Influent | 26.11 (5.20-75.10) | 47.46 (37.65-57.27) | xx | 58.31 (32.46-79.16) | 69.27 (37.97-100.58) | 35.93 (4.96-66.90) | xx |
| | Effluent | 25.84 (10.75-40.93) | 32.86 (17.70-48.01) | xx | 25.40 (18.71-32.09) | 51.25 (29.04-73.46) | 42.46 (10.38-74.55) | xx |
| Total Copper (µg/L) | Influent | 20.14 (8.41-31.79) | 8.91 (5.29-12.52) | 5.65 (2.67-38.61) | 31.93 (25.25-38.61) | 14.57 (10.87-18.27) | 15.42 (9.20-21.63) | xx |
| | Effluent | 12.10 (5.41-18.80) | 6.36 (4.70-8.01) | 4.23 (0.62-7.83) | 10.66 (7.68-13.68) | 10.25 (8.21-12.29) | 14.17 (8.33-20.01) | 2.78 (0.88-8.78) |
| Dissolved Copper (µg/L) | Influent | 6.66 (0.73-12.59) | 7.33 (5.40-9.26) | xx | 14.15 (10.14-18.16) | 7.75 (4.55-10.96) | 13.59 (9.82-17.36) | xx |
| | Effluent | 7.37 (3.28-11.45) | 4.37 (3.73-5.73) | xx | 8.40 (5.65-11.45) | 9.00 (7.28-10.72) | 13.92 (4.40-23.44) | xx |

[1] Actual number of BMPs reporting a particular constituent may be greater or less than the number reported in this table, which was based on number of studies reported in the database based on BMP category.

[2] The biofilter BMP category includes vegetated swales, filter strips, and vegetated buffers.

[3] The media filter BMP category includes sand filters, organic filters, gravel filters, bioretention filters, and proprietary filters.

[4] The hydrodynamic device BMP category includes a wide range of proprietary and non-proprietary hydrodynamic device types, catch basins, and oil/water separators.

Notes: xx - Lack of sufficient data to report median and confidence interval. Values in paranthesis are the 95% confidence intervals about the median. Differences in median influent and effluent concentrations does not necessarily indicated that there is a statistically significant difference between influent and effluent. See *Analysis of Treatment System Performance, International Stormwater BMP Database (1999-2007) (Geosyntec and WWE, 2007) for more detailed information.

*Source: International Stormwater Best Management Practices (BMP) Database June 2008 (www.bmpdatabase.org)*

The following summaries provide interpretation of the BMP Database monitoring data in Table A-1 as well as additional BMP Database results, not included in Table A-1, for the other pollutants of concern for Northern Kentucky (e.g., bacteria/pathogens, organic compounds, oxygen demanding substances, and volume reduction).

Total Suspended Solids

Total suspended solids (TSS) represent the most widely reported storm water constituent in the BMP Database. Each of the BMP categories is designed with sedimentation and/or filtration methods of pollutant removal that will provide treatment for TSS (Table 6.1-3 in the BMP Manual). The best treatment performance commonly occurs with BMPs that provide longer detention periods for sedimentation (e.g., wet ponds) and greater contact area for filtration (e.g., media filters).

Based on the non-parametric statistical analysis, median effluent TSS concentrations are significantly lower than median influent concentrations for wet ponds, biofilters, and media filters. Comparing BMP categories, median effluent concentrations for wet ponds were lowest followed by media filters, porous pavement, wetland basins, biofilters, detention ponds, and hydrodynamic devices. These results generally reflect the performance that would be expected based on the methods of pollutant removal that achieve the greatest removal of TSS. Media filters, porous pavement, and biofilters rely on high filtration capabilities that tend to be effective at TSS removal. Wet ponds and wetland basins rely on high sedimentation (and filtration) capabilities that also tend to be effective for TSS removal.

Information regarding particle size distributions or settling velocities among the studies included in the database is very limited, and no distinction based on these factors is made between BMP studies analyzed. Particle size distribution may play a significant role in BMP performance. For example, coarse sand settles more rapidly than finer particles associated with clayey or silty soils. Although EPA does not provide a national recommended numeric water quality criterion for TSS, many NPDES construction dewatering and wastewater permits identify 30 mg/L as the average permissible TSS concentration. Median concentrations for all of the BMP categories are below 30 mg/L.

Total Phosphorus

Total Phosphorus (TP) is the second most reported constituent in the BMP Database, after Total Suspended Solids (TSS). Many waterbodies in Northern Kentucky are impaired for nutrients/eutrophication (Table 6.1-1 in Chapter 6 of the BMP Manual). Much of phosphorus is associated with particulates; therefore, systems that provide longer detention periods for sedimentation (e.g. wet ponds), and larger contact areas for filtration (e.g. media filters) tend to show better treatment performance for total phosphorus (Table 6.1-3 in Chapter 6 of the BMP Manual). Based on the non-parametric statistical analysis, median effluent TP concentrations are significantly lower than median influent concentrations for hydrodynamic devices, media filters, and wet ponds. These results suggest that sedimentation was the likely method of pollutant removal that significantly reduced TP concentrations for hydrodynamic devices and wet ponds while filtration was the likely method of pollutant removal that significantly reduced TP concentrations for media filters. Median influent TP concentrations are significantly greater than median effluent concentrations for biofilters. The reasons for this are likely due to overfertilization and/or sediment resuspension during larger storms, especially if the biofilter is on-line (i.e., the biofilter is also used as a conveyance channel for flows greater than the water quality design storm). Comparing BMP categories, median effluent concentrations for porous pavement and wet ponds were lowest followed by wetland basins, media filters, detention ponds, hydrodynamic devices, and biofilters. These results generally reflect the performance that would be expected based on the methods of pollutant removal that provide the greatest removal. Porous pavement and media filters have high filtration capabilities for particulate TP while wet ponds and wetland basins have high sedimentation capabilities as well as filtration capabilities.

For additional information on phosphorus refer to Appendix K for the memorandum prepared by LimnoTech for SD1 in 2011, *Nutrient Treatment Efficiency of Bioretention Cells, Constructed Wetlands, and Retention Basins*.

Dissolved Phosphorus

Dissolved Phosphorus (DP) is reported much less frequently in the BMP Database than TP. Many of the impaired waterbodies in Northern Kentucky are impaired for nutrients/eutrophication (Table 6.1-1 in Chapter 6 of the BMP Manual). DP is more difficult to remove than TP due to its dissolved nature. The methods of pollutant removal that are most efficient at removing DP are sorption, ion exchange, chemical precipitation, and uptake by vegetation and microbes (Table 6.1-3 in Chapter 6 of the BMP Manual). Based on the non-parametric statistical analysis, median effluent DP concentrations are significantly lower than median influent concentrations for detention ponds, media filters, and wet ponds. Fewer than five studies were available for the wetland basin category; with more studies, it is likely that wetland basins would show a significant reduction in DP. Median influent DP concentrations are significantly greater than median effluent concentrations for biofilters. The reasons for this are likely due to overfertilization. Comparing BMP categories, median effluent concentrations for wet ponds, media filters, and hydrodynamic devices were lowest followed by detention ponds, wetland basins, and biofilters. These results generally reflect the performance that would be expected based on the methods of pollutant removal that provide the greatest removal. The exceptions are hydrodynamic devices and wetland basins. The primary method of pollutant removal for hydrodynamic devices is sedimentation which is not a primary method of pollutant removal for DP removal; therefore, low DP removal for hydrodynamic devices is expected. The methods of pollutant removal for wetland basins include all of the primary methods of pollutant removal for DP removal; therefore, higher DP removal is often expected for wetland basins than for detention ponds and hydrodynamic devices. As noted previously, fewer than five studies were available for the wetland basin category; therefore, the limited data may not accurately represent wetland basin performance for DP removal. Lack of sufficient data limited reporting of median effluent concentration and confidence interval for porous pavement.

Total Nitrogen

Total nitrogen (TN) includes the total organic (particulate and dissolved) and inorganic (dissolved) forms of nitrogen. Several impaired waterbodies in Northern Kentucky are impaired for nutrients/eutrophication. Among the six BMP categories in the BMP Database, only two categories (biofilters and wet ponds) included more than ten studies reporting TN, which limits comparisons of relative performance across BMP categories. In particular, detention ponds only had three studies reporting TN. Applicable methods of pollutant removal for TN removal include sorption, filtration, microbially mediated transformations (nitrification and denitrification), uptake by vegetation and microbes, and, to a lesser degree, sedimentation (Table 6.1-3 in Chapter 6 of the BMP Manual). Better treatment performance is commonly exhibited for treatment BMPs that provide filtration and microbially mediated transformations (e.g., wetland basins), and more contact area for filtration (e.g., media filters). Based on the non-parametric statistical analysis, all BMP categories except detention ponds and exhibited a significant difference between the median influent and effluent concentrations with hydrodynamic devices exhibiting a significantly higher median effluent concentration as compared to influent. Comparing BMP categories, median effluent concentrations were lowest for media filters and biofilters followed by wetland basins, wet ponds, hydrodynamic devices, and detention ponds.

These results tend to indicate that BMPs with predominate sedimentation methods of pollutant removal (detention ponds and hydrodynamic devices) perform poorly for TN removal as sedimentation generally plays a relatively insignificant role with respect to treatment of nitrogen as compared with microbially mediated transformation, filtration, sorption, and vegetation/microbial uptake methods of pollutant removal that are exhibited by the other BMP categories (TRBNA, 2006). Lack of sufficient data limited reporting of median effluent concentration and confidence interval for porous pavement.

For additional information on nitrogen refer to Appendix K for the memorandum prepared by LimnoTech for SD1 in 2011, *Nutrient Treatment Efficiency of Bioretention Cells, Constructed Wetlands, and Retention Basins*.

Total Kjeldahl Nitrogen

Total Kjeldahl Nitrogen (TKN) represents the sum of organic nitrogen and ammonia. Many waterbodies in Northern Kentucky are impaired for nutrients/eutrophication (Table 6.1-1 in Chapter 6 of the BMP Manual). As a measure of available oxidizable nitrogen, it serves as an indicator of the oxygen that could be consumed through nitrification. It is the most widely reported form of nitrogen in the BMP Database. Applicable methods of pollutant removal for TKN removal are similar to TN and include sorption, filtration, microbially mediated transformations (nitrification and denitrification), uptake by vegetation and microbes, and, to a lesser degree, sedimentation (Table 6.1-3 in Chapter 6 of the BMP Manual). Better treatment performance is commonly exhibited for treatment BMPs that provide filtration and microbially mediated transformations (e.g., wetland basins), and more contact area for filtration (e.g., media filters). For most BMPs in the dataset, the average influent and effluent TKN data exhibited low variability ($Cv < 1$). A significant difference between median influent and effluent TKN concentrations was exhibited for all BMPs except for hydrodynamic devices and wetland basins.

Based on the non-parametric statistical analysis, median effluent concentrations for detention ponds were significantly greater than median influent concentrations. Comparing BMP categories, median effluent concentrations were lowest for wetland basins and wet ponds followed by porous pavement, hydrodynamic devices, biofilters, media filters, and detention ponds. These results indicate that the highest TKN removals occur in systems with longer detention times, and opportunity for microbially mediated transformations; however, wetland basins are often a source of TKN which is the likely result of organic nitrogen in the form of plant matter being exported from the system. Systems that primarily use filtration and sedimentation tend to not do as well at removing TKN as systems that facilitate microbially mediated transformations.

Nitrate Nitrogen

Nitrogen in runoff often takes the form of nitrate nitrogen ($NO_3$ as N), either due to direct export of agricultural or lawn and garden fertilizers and other materials containing high levels of nitrate, or the oxidation of organic and ammonia nitrogen during transport through the watershed. Removal of nitrate nitrogen is primarily through microbially mediate denitrification, where anoxic conditions drive the conversion of oxidized nitrogen (nitrate) to nitrogen gas ($N_2$). Several waterbodies in Northern Kentucky are impaired for nutrients/eutrophication (Table 6.1-1 in Chapter 6 of the BMP Manual). By far the largest number of studies reporting nitrate nitrogen was for biofilters.

Based on the non-parametric statistical analysis, a significant difference between median influent and effluent nitrate concentrations was identified for all BMP categories except hydrodynamic devices and wetland basins (which only had four studies and three studies, respectively). Median effluent concentrations for media filters were significantly greater than influent concentrations. Comparing BMP categories, median effluent concentrations were lowest for wetland basins followed by wet ponds, hydrodynamic devices, detention ponds, biofilters, and media filters although the results exhibited a high degree of variability, and no single category exhibited much lower median effluent values than the others. Because of the dissolved nature of nitrate, systems with longer detention periods generally provide greater potential for microbially mediated transformation (e.g., wetlands and wet ponds). Other systems performed relatively poorly, consistent with their methods of pollutant removal (e.g., filtration, sedimentation) which have less potential for nitrate removal (Table 6.1-3 in Chapter 6 of the BMP Manual). Lack of sufficient data limited reporting of median effluent concentration and confidence interval for porous pavement.

Total Lead

Total lead is the second-most reported metal constituent in the BMP Database after total zinc. Although lead is not a major pollutant of concern in Northern Kentucky, (Table 6.1-1 in Chapter 6 of the BMP Manual), many of the region's land uses (Table 6.1-2 in Chapter 6 of the BMP Manual) are sources of lead, primarily from automobiles. The important forms of metals (lead and other trace metals) from a treatability standpoint are total, dissolved, and particulate- bound metals. If metals are bound to organic or inorganic particulates, viable methods of pollutant removal include sedimentation and filtration either as methods of pollutant removal separate from coagulation/flocculation or in combination with coagulation/flocculation as pretreatment to these operations. If trace metals are present in the dissolved form, sorption and chemical precipitation are the primary methods of pollutant removal (Table 6.1-3 in Chapter 6 of the BMP Manual).

Based on the non-parametric statistical analysis, a statistically significant difference between the median influent and effluent lead concentrations was only exhibited for media filters and wet ponds. All of the BMP categories tended to have lower median effluent concentrations compared to influent. Comparing BMP categories, median effluent concentrations were lowest for wetland basins and media filters followed by wet ponds, biofilters, porous pavement, hydrodynamic devices, and detention ponds. All BMPs generally showed effective removals, with the highest removals in BMPs that provide longer detention for sedimentation and precipitation (e.g., wet ponds and wetland basins) and greater contact area for filtration and sorption (e.g., media filters and biofilters). The poorer performance of detention ponds and hydrodynamic devices reflects the expected performance based on methods of pollutant removal since both of these BMP categories do not exhibit as long of retention times as the other BMP categories for sedimentation and filtration. In addition, neither of these BMPs have significant (if any) contact area for sorption and filtration.

Dissolved Lead

USEPA recommended freshwater criteria for dissolved lead are 65 µg/L (acute) and 2.5 µg/L (chronic) based on 2006 National Recommended Water Quality Criteria. The dissolved lead criteria are a function of hardness in the water column and values presented here correspond to a hardness of 100 mg/L. Effluent concentrations in the dataset were well below the freshwater acute criterion for all but one event mean concentration (EMC) reported for a retention basin. Most median effluent concentrations were also below the chronic criterion with the exception of wet ponds and hydrodynamic devices. Although dissolved lead is not a major pollutant of concern for Northern Kentucky (Table 6.1-1 in Chapter 6 of the BMP Manual), many of the region's land uses (Table 6.1-2 in Chapter 6 of the BMP Manual) are sources of lead, primarily from automobiles. Dissolved lead, like other dissolved metals, is primarily removed through sorption and chemical precipitation (Table 6.1-3).

Based on the non-parametric statistical analysis, a statistically significant difference between median influent and effluent dissolved lead concentrations was exhibited for biofilters, media filters, and wet ponds. Detention ponds, wetland basins, and hydrodynamic devices tended to have greater median effluent concentrations as compared with influent although the difference was not significant. Comparing BMP categories, median effluent concentrations were lowest for wetland basins followed by media filters, biofilters, detention ponds, wet ponds, and hydrodynamic devices. In general, BMPs designed for longer detention times (e.g., wet ponds and wetland basins) allow for greater removal of dissolved metals via chemical precipitation. Although wetland basins tended to have greater median effluent concentrations as compared to influent, the difference was not significant and the median influent concentration for wetland basins was an order of magnitude smaller than the median influent concentrations for the other BMP categories. Media filters and biofilters are also effective for dissolved lead removal because of their large contact area for sorption. Lack of sufficient data limited reporting of median effluent concentration and confidence interval for porous pavement.

Total Zinc

Total zinc, which encompasses both the particulate-borne and dissolved fraction, is one of the most commonly reported metals in the BMP Database. Zinc is particularly prevalent in urban and highway environments, due to atmospheric, industrial, and automobile-related sources and deposition. Tire wear and exposed zinc building materials are thought to be two of the larger sources. Although total zinc is not a major pollutant of concern in Northern Kentucky (Table 6.1-1 in Chapter 6 of the BMP Manual), many of the region's land use are sources of zinc (Table 6.1-2 in Chapter 6 of the BMP Manual). Organic and inorganic bound particulates are primarily removed via sedimentation and filtration with or without coagulation/flocculation as pretreatment. The primary methods of pollutant removal for dissolved metals are sorption and chemical precipitation (Table 6.1-3).

Based on the non-parametric statistical analysis, all BMP categories exhibit a significant difference between the medians of influent and effluent concentrations. Comparing BMP categories, median effluent concentrations were lowest for porous pavement followed by wet ponds, wetland basins, media filters, detention ponds, and hydrodynamic devices. As with total lead, these results generally show that BMPs with longer detention times (e.g., wet ponds and wetland basins) allow for greater sedimentation and precipitation and BMPs with greater contact area (e.g., porous pavement, media filters, biofilters) allow for greater filtration and sorption. The poorer performance of detention ponds and hydrodynamic devices reflects the expected performance based on methods of pollutant removal since both of these BMP categories do not exhibit as long of retention times as the other BMP categories for sedimentation and filtration. In addition, neither of the BMPs have significant (if any) contact area for sorption and filtration.

Dissolved Zinc

Dissolved zinc is reported most frequently in the BMP Database for biofilters and media filters. The wetland basin and porous pavement BMP categories had insufficient data for analysis of median effluent concentrations and confidence intervals. USEPA recommended freshwater chronic and acute criteria for dissolved zinc are both 120 μg/L. Median effluent concentrations for all reported BMP categories were well below this value. Although dissolved zinc is not a significant pollutant of concern in Northern Kentucky (Table 6.1-1 in Chapter 6 of the BMP Manual), many of the region's land use are sources of zinc (Table 6.1-2 in Chapter 6 of the BMP Manual).

Based on the non-parametric statistical analysis, a significant difference between median influent and effluent concentrations was exhibited for all BMP categories with the exception of wet ponds. Hydrodynamic devices exhibited a median effluent concentration that was statistically greater than the median influent concentration while the other reported BMP categories exhibited median effluent concentrations that were statistically less than the median influent concentrations. Comparing BMP categories, median effluent concentrations were lowest for biofilters and detention ponds followed by wet ponds, hydrodynamic devices, and media filters. In general, BMPs designed for longer detention times (e.g., wet ponds and wetland basins) exhibit greater removal of dissolved metals via chemical precipitation and BMPs with larger contact area (e.g. media filters and biofilters) exhibit greater removal of dissolved metals via sorption (Table 6.1-3 in Chapter 6 of the BMP Manual). The results for dissolved zinc generally did not reflect the expected performance based on the primary methods of pollutant removal for dissolved metals removal (i.e., chemical precipitation and sorption). Wet ponds did not show a significant difference between median influent and effluent concentrations although detention ponds did; wet ponds tended to have a higher median effluent concentration than detention ponds although the difference was not significant. The median dissolved zinc concentration for biofilters was reflective of the expected performance as compared to the other BMP categories (i.e., lowest median effluent concentration) although media filters were not (i.e., highest median effluent concentration). The relatively poor performance of hydrodynamic devices was expected based on the methods of pollutant removal for these systems that generally do not provide significant removal of dissolved metals.

Total Copper

Total copper is well-reported in the BMP Database except for wetland basins which have a small number of available studies in the dataset (four studies) limiting conclusions that can be drawn. Although Northern Kentucky waters are not significantly impaired for total copper (Table 6.1-1 in Chapter 6 of the BMP Manual), many of the region’s land use are sources of copper (Table 6.1-2 in Chapter 6 of the BMP Manual).

Based on the non-parametric statistical analysis, a significant difference between median influent and effluent concentrations was identified for biofilters, media filters, wet ponds, and wetland basins. Hydrodynamic devices and detention ponds had lower median effluent concentrations as compared to influent although the difference was not significant. Only median effluent concentrations were provided for porous pavement. Comparing BMP categories, median effluent concentrations were lowest for porous pavement followed by wetland basins, wet ponds, media filters, biofilters, detention ponds, and hydrodynamic devices. As with total lead and zinc, these results generally show that BMPs with longer detention times (e.g., wet ponds and wetland basins) allow for greater sedimentation and precipitation and BMPs with greater contact area (e.g., porous pavement, media filters, biofilters) allow for greater filtration and sorption (Table 6.1-3 in Chapter 6 of the BMP Manual). The poorer performance of detention ponds and hydrodynamic devices reflects the expected performance based on methods of pollutant removal since both of these BMP categories do not exhibit as long of retention times as the other BMP categories for sedimentation and filtration. In addition, neither of the BMPs have significant (if any) contact area for sorption and filtration.

Dissolved Copper

Dissolved copper is not as widely reported in the BMP Database as total copper. The studies reporting the most dissolved copper are for media filters and biofilters. Lack of sufficient data limited reporting of median effluent concentrations and confidence intervals for wetland basins and porous pavement. USEPA freshwater criteria for dissolved copper are 9 µg/L (chronic) and 13 µg/L (acute) based on USEPA 2006 National Recommended Water Quality Criteria. The dissolved copper criteria are a function of hardness in the water column and values presented here correspond to a hardness of 100 mg/L. Although dissolved copper is not a major impairment for water bodies in Northern Kentucky (Table 6.1-1 in Chapter 6 of the BMP Manual), many of the region's land use are sources of copper (Table 6.1-2 in Chapter 6 of the BMP Manual).

Based on the non-parametric statistical analysis, all BMP categories showed a significant difference between median influent and effluent concentrations. Detention ponds, media filters, and hydrodynamic devices appeared to have median effluent concentrations that were significantly greater than influent concentrations. Median effluent concentrations were lowest for wet ponds followed by detention ponds, biofilters, media filters, and hydrodynamic devices. The results for dissolved copper generally reflected the expected performance based on the primary methods of pollutant removal for dissolved metals removal (i.e., chemical precipitation and sorption) (Table 6.1-3 in Chapter 6 of the BMP Manual). The exceptions to this were (1) detention ponds tended to have a lower median effluent concentration than biofilters and media filters, and (2) media filters had a higher median effluent concentration as compared to influent. The relatively poor performance of hydrodynamic devices was expected based on the methods of pollutant removal for these systems that generally do not provide significant removal of dissolved metals.

<u>Mercury</u>

Analyses of BMP performance for mercury removal are not currently available from the BMP Database. The methods of pollutant removal responsible for mercury removal are the same as those for lead, zinc, and copper. Based on the BMP Database performance results for lead, zinc, and copper, it can be inferred that mercury would be maximized by selecting treatment BMPs that (1) facilitate longer detention times (e.g., wet ponds and wetland basins) allowing for greater sedimentation and precipitation and (2) provide greater contact area (e.g., porous pavement, media filters, biofilters) allowing for greater filtration and sorption (Table 6.1-3 in Chapter 6 of the BMP Manual).

<u>Bacteria/Pathogens</u>

Of the impaired water bodies in Northern Kentucky, many are impaired for fecal coliform, an indicator bacteria (Table 6.1-1 in Chapter 6 of the BMP Manual). The BMP Database contains nearly 500 paired fecal coliform monitoring events at 61 sites and more than 100 paired E. coli monitoring events at 12 sites. The following summary of BMP performance for bacteria/pathogens is based on an analysis of the bacteria data in the BMP Database (Clary et. al., 2008). The results indicate that bacteria densities in untreated runoff were consistently high for the majority of the BMP study sites, with the influent densities varying substantially. In addition, the ability of the different BMP categories to reduce bacteria counts varies widely with BMP categories. No single BMP type appears to be able to consistently reduce bacteria in surface effluent to levels below in-stream primary contact recreation standards. As a result, storm water managers, permit writers, and TMDL participants should not assume that structural BMPs can meet numeric effluent limits for bacteria for all storms and under all conditions.

The BMP categories that appear to have potential for bacteria reduction in effluent include wet ponds and media filters. For bacteria reduction, wet ponds may be a suitable BMP option for developments with significant land area. A potential disadvantage of wet ponds is that they can attract waterfowl and wildlife, which can increase bacteria levels. Media filters (inclusive of bioretention areas) show promise in removing bacteria at the site level. The key method of pollutant removal for media filters is filtration. This method of pollutant removal is well proven in the drinking water arena, so it is not surprising that these BMPs would reduce bacteria, provided that the facilities are properly maintained. For existing developments, some targeted retrofitting in bacteria "hot spot" areas could be possible, but costs of watershed-wide retrofits with many media filters will likely be cost prohibitive. Biofilters and detention ponds appear to have low effectiveness in reducing bacteria and in some cases have the potential for exporting bacteria. Potential causes for this include the fact that these BMPs tend to attract ducks, geese, other wildlife, and domestic pets, which may contribute to bacteria loading.

Several BMP categories have datasets that are too small to warrant interpretation; these include porous pavement, and hydrodynamic devices. However, one could anticipate how some of these BMPs may perform by evaluating BMPs with similar methods of pollutant removal. For example, properly designed porous pavement, such as those with a sand layer above the subsurface underdrains should perform similarly to media filters. Hydrodynamic devices do not exhibit methods of pollutant removal that would encourage bacteria reduction; therefore, these BMPs would likely not provide high bacteria removal capabilities. The bacteria-related findings of the BMP Database reinforce earlier research by such investigators as Pitt (2004) and Schueler and Holland (2000).

For additional information on bacteria refer to Appendix K for the memorandum prepared by LimnoTech for SD1 in 2011, *Bacteria Treatment Efficiency of Constructed Wetlands and Retention Basins*.

Toxic Organic Compounds, Pesticides, Oxygen Demanding Substances

Water bodies in Northern Kentucky are not commonly impaired for dioxin and PCBs which are both toxic organic compounds (Table 6.1-1 in Chapter 6 of the BMP Manual). This is also true for low dissolved oxygen and pesticides.

A summary of results for toxic organic compounds, pesticides, or oxygen demanding substances are available in the BMP Database. Although all three pollutant categories are organic compounds, they are differentiated into the three pollutant categories because they originate from different sources and have different impacts on aquatic life and human health. Reporting of the three pollutant categories in the BMP Database is less frequent than other parameters such as TSS, nutrients, and metals. In addition, these pollutant categories are often measured as a variety of different parameters, especially toxic organic compounds and pesticides that can occur as hundreds of different individual parameters making analyses less practical. The most frequent parameter in the database for quantifying toxic organic compounds is total organic carbon (TOC) although pesticides and oxygen demanding substances are also included in the TOC measurement. Oxygen demanding substances are biodegradable organic compounds (e.g., sewage) that are usually measured as biochemical oxygen demand (BOD) or chemical oxygen demand (COD).

For the purposes of this paper, an evaluation of expected performance for removal of the three organic pollutant categories will be provided here for the different BMP categories based on the methods of pollutant removal that target organic compound removal. Toxic organic compounds and pesticides are primarily removed through sorption, microbially mediated transformation, plant/microbe uptake and storage, aeration/volatilization. The primary BMP categories that remove pollutants based on sorption are media filters, biofilters, porous pavement, and infiltration BMPs (i.e. vegetated swales, filter strips, bioretention, etc.). The primary BMP categories that remove pollutants based on microbially mediated transformation, plant/microbe uptake and storage, and aeration/volatilization are wet ponds, wetland basins, and detention ponds. Oxygen demanding substances are primary removed through microbially-mediated transformations. The primary BMP categories that target microbially-mediated transformations are wet ponds, wetland basins, and detention ponds.

Volume Reduction

Although volume reduction is not a specific pollutant of concern as identified in Tables 6.1-1 and 6.1-2 in Chapter 6 of the BMP Manual, the impacts of runoff volume increase are ubiquitous for urban development. Low impact development techniques and treatment BMPs that achieve volume reduction can contribute significantly towards addressing peak flow and assisting post-construction developments mimic predevelopment hydrology.

The BMP Database also includes datasets to assess the volume reduction potential of the different BMP categories. Results shown in Table A-2 indicate that certain BMP types may reduce the volume of runoff through evapotranspiration and/or infiltration (Strecker et. al., 2004). On average, detention ponds were found to reduce runoff volumes by an average of 30% (comparison of inflow volume to outflow volume), while biofilters reduced volumes by almost 40%. As expected, wet ponds, wetland basins, and hydrodynamic devices showed little or no runoff volume reductions. Media filters also did not show a reduction in runoff volume. Most of the media filters in the database are contained or lined reducing the potential for infiltration (e.g., sand filters, cartridge filters). Bioretention areas are a particular exception although very few bioretention area studies are currently available in the BMP Database. A study of three field sites in North Carolina showed that unlined bioretention areas can reduce total outflow by 50%, even in clayey soils (Hunt et. al., 2006). As more bioretention area studies are added to the database, it is likely that the media filter category will show some runoff volume reduction or a separate BMP category will be created. Since the BMP Database currently does not have enough data to separate out bioretention areas from other media filters, a separate performance evaluation for bioretention areas is provided in a separate section below and is based on current a literature review of bioretention area studies.

Table A-2 Ratio of Mean Monitored Storm Event Outflow to Inflow for Storms Greater Than 0.2 Watershed Inches.

| BMP CATEGORY | MEAN MONITORED OUTFLOW/MEAN MONITORED INFLOW FOR EVENTS WHERE INFLOW IS GREATER THAN OR EQUAL TO 0.2 WATERSHED INCHES |
|---|---|
| Detention Ponds | 0.70 |
| Wet Ponds | 0.93 |
| Wetland Basins | 1.00 |
| Biofilters | 0.62 |
| Media Filters | 1.00 |
| Hydrodynamic Devices | 1.00 |
| Table excerpted from Strecker et. al., 2004. | |

Although not provided in the BMP database (Table A-2), other BMP categories can also significantly reduce runoff volumes. Porous pavement studies conducted by North Carolina State University (NCSU) showed that permeable pavement sites reduced runoff volume by at least 60% (Hunt and Szpir, 2006). More recent studies from NCSU show even greater volume reduction potential (>90%) for permeable pavement (Hunt and Collins, 2008). In addition, insufficient data is available in the database for assessing the performance of infiltration BMPs (i.e. vegetated swales, filter strips, bioretention, etc.), green roofs, and retention BMPs (i.e. wet ponds, permeable pavement). For infiltration BMPs, it can be assumed that 100% of the inflow volume (i.e., capture volume) to infiltration BMPs is reduced through infiltration as long as the BMPs are properly maintained. For green roofs, studies have shown that over 50% of annual runoff can be retained (Hunt and Szpir, 2006). Similar assumptions can be made for retention BMPs as infiltration BMPs (i.e. vegetated swales, filter strips, bioretention, etc.). If stored runoff will be used for irrigation, it can be assumed that 100% of the inflow volume (i.e., capture volume) is reduced through infiltration. If stored runoff is used for other non-potable uses (e.g., laundry water, toilet flushing), it is assumed that this volume is not reduced since it is returned to receiving waters via the wastewater treatment plant. If septic systems are used, then the stored runoff volume used for non-potable water uses would be considered reduced as it infiltrates through the systems leach field.

Based on the volume reduction potential for biofilters, detention ponds, bioretention areas, porous pavement, infiltration BMPs, and green roofs, there is a basis for factoring in volume and resulting pollutant load reductions into BMP performance. This has significant implications for Total Maximum Daily Load (TMDLs) implementation planning and other storm water management planning. It is also expected that as BMPs that are specifically designed to reduce runoff volumes (e.g., low impact development BMPs) are tested and data added into the BMP Database, that new BMP categories will be included and volume reduction results will improve (Strecker et. al., 2004).

Additional Bioretention Performance Studies

Currently, the BMP Database only contains three studies characterizing the performance of bioretention areas. These three studies are combined into the BMP category of media filters. Due to lack of data in the BMP Database, a supplemental review of national studies is provided here to assist in characterizing the performance of bioretention areas. Insufficient data was available in the literature to evaluate bioretention performance based on median effluent concentrations and therefore, performance is presented based on percent removal. It should be noted that studies have shown that effluent quality, rather than percent removal, may be more reliable in assessing storm water treatment BMP performance, as percent removal is more or less a function of how "dirty" the inflow is (Strecker et. al., 2001). For example, high pollutant concentrations in the BMP influent may allow for high removal efficiencies, while the same BMP may produce very low removal efficiencies when influent concentrations are low. However, the majority of bioretention studies available report performance based on percent removal. Table A-3 provides a summary of median percent removals based on the literature review of national studies for specific pollutants of concern.

Table A-3 Bioretention Performance Evaluation Based on Literature Review

| POLLUTANT OF CONCERN | PERCENT REMOVAL[1,2] |
|---|---|
| TSS, mg/L | 86% |
| Total Phosphorus, mg/L | 68% |
| TKN, mg/L as N | 60% |
| Total Copper, µg/L | >90% |
| Total Lead, µg/L | >90% |
| Total Zinc, µg/L | >90% |
| Volume Reduction, outflow volume/ inflow volume | >50% |

1 The percent removal values were estimated based on performance results obtained from the following studies: Ballestro et. al., 2005; Davis et. al., 2006; Davis et. al. 2003; Davis et. al, 2001; Hseih et. al., 2005a; Hsieh et. al., 2005b; Hunt et. al., 2006; Hunt and Lord, 2006; Kim et. al., 2003; Sun and Davis, 2007.

2 Many of the bioretention studies used to estimate percent removal in Table A-3 were lab scale and bench scale studies. This differs from the BMP Database which does not allow lab studies. The estimated percent removal presented here likely overestimates the actual percent removal that would be observed in field scale studies. Also noting that percent removal is not as representative of actual BMP performance as effluent quality.

Based on national studies, bioretention areas provide relatively consistent and high pollutant removal for TSS and metals. Most of the TSS removal occurs in the top mulch layer while metals removal commonly occurs within the first 18 inches of the soil media (Hseih and Davis, 2005a; Hunt and Lord, 2006). Removal of nitrogen and phosphorus species is less consistent but studies have identified specific design criteria that can reliably improve performance for these constituents (Kim et. al., 2003; Hseih and Davis, 2005a; Hunt et. al., 2006; Hunt and Lord, 2006). While bioretention performance reported herein is based on national studies, two factors suggest that these BMPs will be successful in field applications in Northern Kentucky. First, the material that makes up the treatment media in a bioretention area does not depend on native soil; rather, it is specified and imported. Second, the method of pollutant removal required to treat the pollutants of concern will be present in properly designed bioretention areas regardless of regional climate. In addition, bioretention areas have shown to have high volume reduction potential. A study of three field sites in North Carolina showed that unlined bioretention areas can reduce total outflow volume by 50%, even in clayey soils (Hunt et. al., 2006).

Additional Performance Evaluation for Hydrodynamic Devices

As described above, the hydrodynamic device BMP category in the BMP Database represents a wide range of various proprietary and non-proprietary device types with performances that vary significantly. Hydrodynamic devices target trash and debris, sediments, and oil and grease. This section provides an additional evaluation of hydrodynamic devices specifically focusing on the performance of specific proprietary hydrodynamic devices based on scientific third-party evaluations. The Vortech System by Vortechnics, Inc. and the BaySaver Separator Unit by BaySaver Technologies, Inc. both underwent Environmental Technology Verification (ETV) testing by the USEPA. Performance for these two products is summarized in terms of average effluent quality as well as removal efficiency (i.e., percent removal). It should be noted here that studies have shown that effluent quality, rather than percent removal, may be more reliable in assessing storm water treatment BMP performance. Table A-4 below provides a summary of performance data for the two ETV tested proprietary hydrodynamic devices. The ETV study of the Vortech System was conducted in Wisconsin and the ETV study of the BaySaver Separator System was conducted in Georgia. Both studies received full QA/QC as required by the ETV program.

Table A-4 Performance Evaluation For Two Hydrodynamic Device Types

| POLLUTANT | VORTECH SYSTEM[1] | | | BAYSAVER SEPARATOR SYSTEM[1] | | |
|---|---|---|---|---|---|---|
| | MEDIAN OF INFLUENT EMCs | MEDIAN OF EFFLUENT EMCs | MEDIAN PERCENT REMOVAL[2,3] | MEDIAN OF INFLUENT EMCs | MEDIAN OF EFFLUENT EMCs | MEDIAN PERCENT REMOVAL [2,3] |
| TSS, mg/L | 104 | 76.5 | 32% | 26 | 33 | -24% |
| Total Phosphorus, mg/L | 0.16 | 0.14 | 15% | 0.17 | 0.15 | 13% |
| Nitrate, mg/L as N | N/A | N/A | N/A | 0.41 | 0.33 | 18% |
| TKN, mg/L as N | N/A | N/A | N/A | 1.3 | 1 | 20% |
| Total Copper, µg/L | 53.5 | 37.5 | 15% | 15 | 20 | 0% |
| Dissolved Copper, µg/L | 14 | 16 | -8% | N/A | N/A | N/A |
| Total Lead, µg/L | N/A | N/A | N/A | 50 | 90 | -50% |
| Dissolved Lead, µg/L | N/A | N/A | N/A | N/A | N/A | N/A |
| Total Zinc, µg/L | 240 | 170 | 28% | 140 | 100 | -5% |
| Dissolved Zinc, µg/L | 56.5 | 69 | -18% | N/A | N/A | N/A |

1 From ETV Report (USEPA, 2005)

2 Percent removal expressed as a median value from paired datasets; therefore, values may not correspond specifically with median influent and effluent values (i.e., difference between median influent and effluent concentrations may suggest a percent removal greater or lesser than the median percent removal)

3 Negative percent removal indicates a production or release of pollutants N/A indicates data not available, not sufficient to calculate statistics and/or not applicable

Based on results of the BMP Database and the ETV studies (Table X-4), hydrodynamic devices, in general, exhibit marginal performance for addressing the pollutants of concern for Northern Kentucky. Vortech System showed the best median percent removal for TSS (32%) of the two hydrodynamic devices studied. As additional verification of this assessment, a study conducted by the University of New Hampshire Stormwater Center compared the performance of several proprietary hydrodynamic devices including Aqua-Swirl and Aqua-Filter, VorSentry, V2B1 Structural System, and Continuous Deflective Separation (CDS) Unit. Results of the New Hampshire study also suggest that the water quality performance of hydrodynamic devices is moderate to poor. Aqua-Swirl and Aqua-Filter showed the best median percent removal for TSS (66%) of the hydrodynamic devices studied. VortSentry showed the lowest median percent removal for TSS (29%) (Ballestro et. al, 2005).

Hydrodynamic devices tend to be most effective when used for pretreatment in areas where runoff is expected to contain sediment particles greater than 100 microns in diameter (Roseen et. al., 2007).

Therefore, hydrodynamic devices are most applicable as a pretreatment BMP or if other BMP types are prohibitive as a result of space constraints.

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# APPENDIX B
## BIORETENTION SOIL MIX

Structural Best Management Practice Component Specification

*Applications*

- Bioretention areas / Rain gardens
- Planter boxes
- Swales
- Filter strips

*Advantages*

- ✓ Rapid drainage
- ✓ Proper nutrient/organic content
- ✓ Slow release of nutrients to avoid leaching
- ✓ Proper pH

These specifications are to be used as guidance when bioretention soil mix is called out in bioretention areas, planter boxes, swales, filter strips, or other vegetated BMPs. This soil mix is designed to create a soil that will drain well and has proper nutrient/organic content without leaching nutrients to underdrains, should they be used.

It is important to remember that no bioretention soil mix will be appropriate for all circumstances. Therefore, modifications will be needed depending on particulars of each use; the information below is provided as guidance. Some specific applications requiring modification include:

Phosphorus removal applications – For situations in which high phosphorus is a problem, the P index of the soil used should be checked to ensure it is between 10 and 30. Use of a soil with a high P index can actually release phosphorus into the storm water that passes through it. In applications where phosphorus is not an issue, a P index of 25 – 40 is appropriate (Hunt and Lord, 2006).

Nitrogen removal applications – Nitrogen removal requires longer contact time, so soils should be mixed to reduce the hydraulic conductivity to approximately 1 inch/hour. Soil thickness should also be increased in these applications. Usually, 36 inches of soil is preferred.

Specific hydraulic conductivity – This is primarily affected by the percentage of fines (i.e., particles which pass a No. 200 sieve and include silt-sized and clay-sized particles) that are part of the soil mix. While there is a range of reasonable values, high permeability soils should be closer to 4% fines, while lower permeability should be closer to 12% fines.

## SPECIFICATIONS

The following specifications and guidance are provided for general bioretention soil mixes with relatively high hydraulic conductivity. As mentioned above, individual BMPs may vary depending upon the specific application.

- The planting media shall be highly permeable and high in organic matter (e.g., loamy sand mixed thoroughly with compost amendment) and a surface mulch layer.
- Planting media shall consist of 70 - 80% sand, 10 - 15% compost, and 10 to 20% clean topsoil with 98% of the media (by volume) passing through a 3/4" sieve (or screen). The organic content of the soil mixture should be 4 - 8%; the pH range should be 5.5 to 7.5.
- Sand should be free of stones, stumps, roots or other similar objects larger than 5 millimeters, and have the following gradation:

| PARTICLE SIZE (ASTM D422) | % PASSING |
|---|---|
| #4 | 100 |
| #6 | 88-100 |
| #8 | 79-97 |
| #50 | 11-35 |
| #200 | 5-15 |

- Compost shall be free of stones, stumps, roots or other similar objects larger than ¾ inches, have a particle size of 98% passing through ¾" screen or smaller and shall have dark brown color and a soil-like odor. Compost exhibiting a sour or putrid smell, containing recognizable grass or leaves, or is hot (120 F) upon delivery or rewetting is not acceptable. Compost shall also meet the following characteristics:
    - Soluble Salt Concentration: < 8 mmhos/cm (dS/m)
    - pH: 6.0-8.0
    - Moisture: 30-60% wet weight basis
    - Organic matter: 35-65% dry weight basis
    - Physical contaminants: < 1% dry weight basis

- Topsoil shall be free of stones, stumps, roots or other similar objects larger than 2 inches, and have the following characteristics:

| PARTICLE SIZE (ASTM D422, D1140) | % PASSING |
|---|---|
| 3/4" | 98 |
| Sand (0.05 - 2.0 mm ) | 50-75 |
| Silt (0.002 - 0.05 mm) | 15-40 |
| Clay | < 5 |

    - Soluble salts: < 4.0 mmhos/cm (dS/m)
    - pH range: 5.5 to 7.0
    - Organic matter: > 4%
    - Carbon to nitrogen ratio: < 25:1
    - Moisture content: 25-55%

- The bioretention soil shall be covered with 2 – 3 inches of mulch at the start and an annual placement of 1-2 inches of mulch. *Intent: this will help sustain nutrient levels, suppress weeds, and maintain infiltrative capacity.* Mulch shall be:

- Well-aged, shredded or chipped woody debris or plant material. Well-aged mulch is defined as mulch that has been stockpiled or stored for at least twelve (12) months. Compost meeting the requirements above may also be used (compost is less likely to float and is a better source for organic materials).
- Free of weed seeds, soil, roots and other material that is not bole or branch wood and bark.
- A maximum of 2 to 3 inches thick. *Intent: thicker applications can inhibit proper oxygen and carbon dioxide cycling between the soil and atmosphere.*
- Grass clippings or pure bark shall not be used as mulch.

- Planting media design height shall be marked appropriately, such as a collar on the vertical riser (if installed), or with a stake inserted 2 feet into the planting media and notched to show bioretention surface level and ponding level.

- The soil mix shall be tested and meet the following criteria:

| ITEM | CRITERIA | TEST METHOD |
|---|---|---|
| Corrected pH | 5.5 – 7.5 | ASTM D4972 |
| P – index | 10 – 40 | Mehlich 3 |
| Hydraulic conductivity | 6 – 10 in/hr | ASTM F1815 |
| Soluble salts | Not to exceed 500 ppm | * |

* Use authorized soil test procedures.

*Note: Hydraulic conductivity is equivalent to permeability. Infiltration rate is equal to the hydraulic conductivity multiplied by a hydraulic gradient, which is a function of the head pressure. When submitting soil testing results, documentation should indicate if values are representative of hydraulic conductivity or infiltration rates.*

## ADDITIONAL SOURCES OF INFORMATION


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# APPENDIX C
## PLANT SELECTION

The Plant Selection Table, in this appendix, is provided as a tool that can be used in the identification of plant species that may be appropriate for the location, size and environmental characteristics of the site. These plants represent a small sample of plants that are available, and have been chosen with an emphasis on native species appropriate for use in BMPs in Northern Kentucky. Native plants are recommended for storm water BMPs because they are more tolerant of local climates, soils and water conditions, and their deep rooting structure enhances water uptake and soil permeability.

To select plants, start by identifying the moisture conditions of the planting area; this information can then be used to identify the plants that meet this critical site requirement. Moisture requirements for each plant are listed in the Plant Selection Table, varying from Wet Mesic to Dry conditions; the moisture requirements correspond to the drainage capacity of the soils. The term Wet Mesic refers to soils that are usually very moist for most of the growing season and have poor infiltration. Mesic soils typically have good drainage and remain moist during the growing season. Dry soils vary from somewhat moist to very dry during the growing season. In addition to soils, the slope and exposure of a site can cause significant variations in the moisture that is available to plants within a small area. These variations must also be considered when identifying and locating plants. Next, consider the amount of sunlight that will be available for the proposed feature. Sunlight requirements for each plant are listed in the Plant Selection Table, ranging from full sun to partial sun and shade.

After identifying the basic site parameters, the next step is to consider the size of the plants that are desired for the planting area. Plant sizes should be considered based on the overall size and layout of the BMP feature, reserving the taller selections for larger applications. In general taller plants should be located in the center of the feature or they may be used as a backdrop. With these tasks completed, the aesthetic aspects of the feature become the primary focus in further refinement of the plant list. Especially large sites may require the addition of larger plant materials including native shrubs and trees.

Design elements such as plant color and texture as well as bloom color and bloom time are the next plant criteria to be considered. Color selection can focus on a theme such as primarily yellow flowers or a composite of a variety of colors selected from the plants that meet the basic site requirements. The texture of plants is an often overlooked characteristic that should be considered. A variety of textures can add visual depth and interest in the feature.

The final planning stage involves the layout or planting plan. A sketch or drawing of the planted area should be developed so areas can be identified for each plant type. Plants should be placed by grouping individual species in groups of three to fifteen plants, depending on the size of the feature. Space individual plants at 12” to 18” on center. Plant groupings will provide a statement of color and texture. When locating the plant groups, consider plant texture, size, color and bloom time relative the adjacent plants. Groups should be repeated to create cohesion of the plan. Provide a diverse mixture of sedges, rushes, and grasses with your flowering species to enhance the diversity and viability of the BMP. Sedges, rushes and grasses can be easily identified in the plant table by their green photo border and the light background in the corresponding row (i.e. Big Bluestem Grass).

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# PLANT SELECTIONS

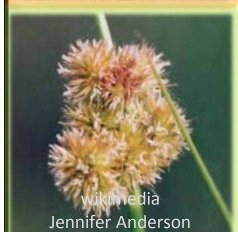
| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Scientific Name | Common Name | Wet-Mesic, Mesic, Dry-Mesic, Dry | Full - Partial - Shade - | Max.Height (inches) | Soil PH | April | May | June | July | Aug | Sept | Oct |
|  | Acorus calamus | Sweet Flag | WM | Full | 24" | 5.2-7.2 | | M | J | J | | | |
|  | Andropogon gerardii | Big Bluestem Grass | WM, M, DM, D | Full Partial | 84" | 6.0-7.5 | | | | | | | |
|  | Asclepias incarnata | Swamp Milkweed | WM, M | Full | 48" | 5.0-8.0 | | | J | J | A | | |
|  | Aster laevis | Smooth Blue Aster | WM, M, DM | Full Partial | 48" | 5.8-7.8 | | | | | A | S | O |
|  | Symphyotrichum novae-angliae | New England Aster | WM, M, DM | Full Partial | 48" | 5.1-6.5 | | | | | A | S | O |

# PLANT SELECTIONS

| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Scientific Name | Common Name | Wet-Mesic, Mesic, Dry-Mesic, Dry | Full - Partial - Shade - | Max.Height (inches) | Soil PH | April | May | June | July | Aug | Sept | Oct |
|  | Baptisia australis | Blue Wild Indigo | WM, M | Full Partial | 48" | 6.1-7.5 | | M | J | J | | | |
|  | Calamagrostis canadensis | Blue Joint Grass | WM, M | Full Partial | 48" | 4.5-8.0 | | | | | | | |
|  | Caltha palustris | Marsh Marigold | WM | Full Partial Shade | 24" | 4.9-6.8 | A | M | J | | | | |
|  | Carex hystericina | Porcupine Sedge | WM | Full | 36" | 6.5-7.5 | | | | | | | |
|  | Carex vulpinoidea | Fox Sedge | WM, M, DM | Full Partial | 42" | 6.8-8.9 | | | | | | | |

| | PLANT SELECTIONS | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
| | Scientific Name | Common Name | Wet-Mesic, Mesic, Dry-Mesic, Dry | Full - Partial - Shade - | Max.Height (inches) | Soil PH | April | May | June | July | Aug | Sept | Oct |
|  | Chelone glabra | White Turtlehead | WM | Full | 60" | 5.1-6.5 | | | | J | A | S | |
|  | Echinacea Purpurea | Purple Coneflower | WM, M, DM | Full Partial | 48" | 6.5-7.2 | | | | J | A | S | |
|  | Eryngium yuccifolium | Rattlesnake Master | WM, M, DM | Full | 48" | 6.6-7.5 | | | | J | A | S | |
|  | Eupatorium maculatum | Joe Pye Weed | WM | Full Partial | 60" | 6.1-7.8 | | | J | J | A | | |
|  | Eupatorium perfoliatum | Boneset | WM | Full Partial | 48" | 6.1-7.8 | | | | J | A | S | |

# PLANT SELECTIONS

| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Scientific Name | Common Name | Wet-Mesic, Mesic, Dry-Mesic, Dry | Full - Partial - Shade - | Max.Height (inches) | Soil PH | April | May | June | July | Aug | Sept | Oct |
|   | Glyceria striata | Fowl Manna Grass | WM, M | Full Partial Shade | 36" | 4.0-8.0 | | | | | | | |
|   | Helenium autumnale | Sneezeweed | WM | Full Partial | 48" | 4.0-7.5 | | | | | A | S | O |
|   | Helianthus grosseserratus | Saw-tooth Sunflower | WM, M, DM | Full Partial | 96" | 5.8-7.3 | | | | | A | S | O |
|   | Iris virginica | Blue Flag Iris | WM, M | Full Partial | 36" | 4.8-7.3 | | M | J | J | | | |
|   | Juncus torreyi | Torrey's Rush | WM, M | Full | 12" | 4.5-6.5 | | | | | | | |

## PLANT SELECTIONS

| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Scientific Name | Common Name | Wet-Mesic, Mesic, Dry-Mesic, Dry | Full - Partial - Shade - | Max.Height (inches) | Soil PH | April | May | June | July | Aug | Sept | Oct |
|   | Liatris pycnostachya | Prairie Blazing Star | WM, M | Full Partial | 48" | 6.0-8.5 | | | | J | A | S | |
|   | Liatris spicata | Marsh Blazing Star | WM, M | Full Partial | 60" | 5.6-7.5 | | | | J | A | S | |
|   | Lobelia cardinalis | Cardinal Flower | WM | Full Partial | 48" | 5.8-7.8 | | | | J | A | S | |
|   | Lobelia siphilitica | Great Blue Lobelia | WM, M | Full Partial | 36" | 6.1-7.8 | | | | J | A | S | O |
|   | Lycopus americanus | Water Horehound | WM | Full | 24" | 5.2-7.8 | | | | J | A | S | |

## PLANT SELECTIONS

| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Scientific Name | Common Name | Wet-Mesic, Mesic, Dry-Mesic, Dry | Full - Partial - Shade - | Max.Height (inches) | Soil PH | April | May | June | July | Aug | Sept | Oct |
|   | Mimulus ringens | Monkey Flower | WM | Full Partial | 24" | 5.6-7.5 | | | J | J | A | S | |
|   | Monarda fistulosa | Wild Bergamot | WM, M, WM, D | Full Partial | 48" | 6.0-8.0 | | | | J | A | S | |
|   | Panicum virgatum | Switch Grass | WM, M, DM, D | Full Partial | 48" | 4.5-8.0 | | | | | | | |
|   | Penstemon digitalis | Foxglove Beardtongue | M, DM | Full Partial Shade | 48" | 5.5-7.0 | | | J | J | | | |
|   | Physostegia virginiana | Obedient Plant | WM, M | Full Partial | 48" | 5.6-7.5 | | | | | A | S | |

| | PLANT SELECTIONS | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
| | **Scientific Name** | **Common Name** | **Wet-Mesic, Mesic, Dry-Mesic, Dry** | **Full - Partial - Shade -** | **Max.Height (inches)** | **Soil PH** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** |
|  | Rudbeckia laciniata | Wild Golden Glow | WM, M | Full Partial Shade | 84" | 4.5-7.0 | | | | J | A | S | O |
|  | Scirpus validus | Great Bulrush | WM | Full | 72" | 5.4-7.5 | | | | | | | |
|  | Silphium perfoliatum | Cup Plant | WM, M | Full Partial | 96" | 4.5-7.5 | | | | J | A | S | |
|  | Solidago ohioensis | Ohio Goldenrod | WM, M | Full | 36" | 5.6-7.5 | | | | J | A | S | |
|  | Spartina pectinata | Prairie Cord Grass | WM, M | Full Partial | 96" | 6.0-8.5 | | | | | | | |

| | PLANT SELECTIONS | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Moisture | Sunlight | Height | | Bloom Time/Bloom Color | | | | | | |
| | **Scientific Name** | **Common Name** | **Wet-Mesic, Mesic, Dry-Mesic, Dry** | **Full - Partial - Shade -** | **Max.Height (inches)** | **Soil PH** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** |
|   | Verbena hastata | Blue Vervain | WM, M | Full Partial | 60" | 5.6-7.5 | | | | J | A | S | |
|   | Verbena stricta | Hoary Vervain | DM, D | Full Partial | 24" | 5.6-7.5 | | | J | J | A | S | |
|   | Veronicastrum virginicum | Culver's Root | WM, M, DM | Full Partial | 60" | 6.6-7.8 | | | J | J | A | | |
|   | Zizia aurea | Golden Alexanders | SM, M, DM | Full Partial | 36" | 6.1-7.8 | A | M | J | | | | |

**Plant Selection Tips**

- Start by considering moisture conditions and the variation of moisture regimes within the area to be planted.
- Consider size and scale of the area to be planted avoiding taller plants in small rain gardens. Arrange taller plants near the middle of the rain garden.
- Select smaller more ornamental species for small urban rain gardens. Arrange plants in groups if a more refined look is desired.
- Consider plant sunlight requirements, soil and site conditions of the proposed rain garden.
- Select at least two plants from the categories of (Grass, Sedge or Rush). Additional plants may need to be considered including native shrubs and trees.
- Consider plant bloom time,color and texture.

# APPENDIX D
# SITE SOIL TYPE AND INFILTRATION TESTING

## PURPOSE OF SOIL AND INFILTRATION TESTING

The purpose of site soil and infiltration testing is to evaluate the condition of the soils and determine the in situ infiltration rate at the location where structural treatment BMPs are proposed to be located. A preliminary site soil assessment is recommended to identify candidate BMP sites that are most amenable to infiltration. This section summarizes the methods for conducting (1) subsurface soil investigations and (2) infiltration testing at candidate infiltration testing locations identified in preliminary site assessments.

A qualified soil scientist or geotechnical professional should conduct the subsurface soil investigation and infiltration tests. The professional should be experienced with the testing procedures as well as the hydraulic functioning of the potential BMPs to ensure that necessary additional information related to BMP siting is acquired during the subsurface soil investigation and infiltration tests.

This appendix is not intended to be applied as a protocol for conducting soil and infiltration testing. Instead, this section is provided to assist in specifying and standardizing soil and infiltration testing techniques across sites within Northern Kentucky where development is occurring.

## SUBSURFACE SOIL INVESTIGATIONS

A subsurface soil investigation is an important part of assessing site soil conditions. Soil maps and hydrologic soil groups are based on regional data and provide only a general understanding of what to expect; however, there are undoubtedly unknowns that will be discovered during these initial field observations. A subsurface soil investigation involves drilling test borings and/or excavating test pits. Both test borings and test pits allow for locally assessing the soil conditions as they change with depth. Series of test borings and test pits enable an evaluation of how the soil conditions change horizontally. In an individual test pit, the variation in localized soil conditions can be observed vertically and horizontally in addition to the soil horizons. To maximize the knowledge gained during the subsurface soil investigation, field tests and observations should be conducted during this process. Additionally, samples of the soils encountered should be recovered for laboratory testing if needed (e.g., gradation analyses, Atterberg limits testing, natural density and moisture content determinations, etc.).

Test borings or test pits are recommended to extend to a depth of at least 3 feet deeper than the proposed bottom of the BMP. If the BMP is intended to infiltrate the entire design storm, the test borings or test pits are recommended to be excavated to a depth of at least 10 feet deeper than the proposed bottom of the BMP. In general, test borings would be considered advantageous over test pits when the depth exceeds 5 feet from existing grades; otherwise, the test pit excavation would become significantly large to accommodate appropriate OSHA regulations for safety. A project that imports fill should characterize the proposed soil profile at the specified depths. For example, if the proposed thickness of fill is 5 feet above existing grade and an infiltration BMP is to be used in the location of the fill, both the fill and the in situ subsoil require soil characterization. Figure D-1 illustrates the proposed soil profile that would result with a 5-foot grade change (i.e., 3 feet of fill overlain by 2 feet of the infiltration BMP). Since the test boring or test pit should be advanced to a depth that is 10 feet deeper than the bottom of the proposed infiltration BMP, a subsurface soil investigation of the top 7 feet of the in situ subsoil would be recommended, in addition to the laboratory sample of the proposed fill material. Characterization of the fill material should be conducted in a laboratory. Additionally, it is recommended that soil compaction is limited in the location of a proposed infiltration BMP.

As the subsurface soil investigation is performed, the following measurements should be made:

- Standard penetration testing (SPT) in the test borings, and

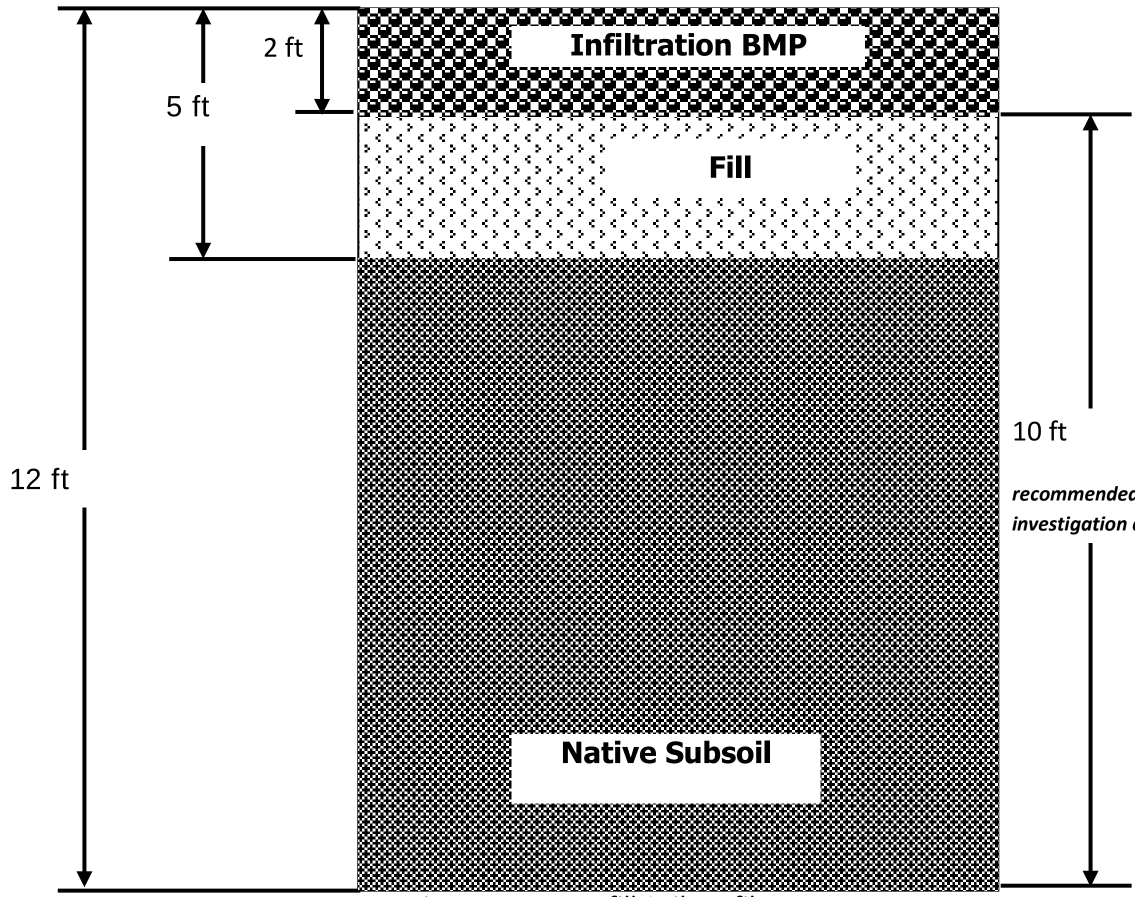


Figure D-1: Post-fill Soil Profile

- Infiltration testing with at least one test occurring at the proposed bottom of the BMP and one test occurring between 3 and 10 feet below the bottom of the infiltration BMP (depth to be determined by the percentage of the design storm that is to be infiltrated into the in situ soils).

Specifically for test pits, the following observations should be made:

- Elevation of groundwater table or indications of the seasonally high groundwater table should be noted using the National Resources Conservation Service (NRCS) hydric soil field indicators guide (NRCS, 2003);

- Soil horizon observations, including: depths indicating upper and lower boundaries of the soil horizons, depths to limiting layers (i.e., layers with low permeability such as bedrock and clay), soil textures, colors and their patterns, and estimates of the type and percent of coarse fragments;
- Locations and descriptions of macropores (i.e., pores and roots); and
- Other pertinent information/observations

For test borings, the soil should be characterized by the recovered soil samples. Soil samples should also be recovered where necessary from the different soil layers in test pits.

The number of test borings and test pits excavated per site depends largely on the specific site and the proposed development plan. Additional tests are recommended if local conditions indicate significant variability in soil types, geology, water table elevations, bedrock, topography, etc. Similarly, uniform site conditions may indicate that fewer test pits are required. Excessive testing and disturbance of the soil prior to construction is not recommended. When the subsurface soil investigations are complete, including infiltration testing, the boreholes and pits should be backfilled with the original in situ soil and the surface replaced with the original topsoil.

## INFILTRATION TESTING

There are a variety of infiltration field test methodologies available to determine the infiltration rate of a soil. Infiltration tests should be conducted in the field in order to ensure that the measurements are representative of actual site conditions (including inherent heterogeneity). As previously mentioned, it is recommended that infiltration rates should be determined at a minimum of two locations in each test pit, with at least one conducted at the proposed bottom depth of the BMP. The actual number of infiltration tests required depends on the soil conditions and the desired function of the BMP. For instance, if the soils are highly variable across the proposed site, more tests are recommended. Also, if the function of the BMP is to infiltrate at a relatively fast rate and the in situ soils are characterized as relatively impermeable (e.g., clay or bedrock), fewer tests may be needed.

For BMPs that infiltrate water through the surface soil layer (e.g., bioretention areas, permeable pavement), choosing a method that measures infiltration in surface soils is important. For subsurface vaults where infiltration will occur at a greater depth in the soil matrix, borehole methods may be more appropriate. Depending on the type of infiltration BMP and depth at which the infiltration test should be conducted, there are several types of infiltration tests that can be used including: disc permeameters, single- and double-ring infiltrometers, and borehole permeameters.

Disc permeameters are typically used to provide estimates of the permeability of soils near saturation but can prove difficult due to required measurements of three-dimensional flow. This device is also commonly used for assessing infiltration rates of already constructed permeable pavements and is generally not used for assessing infiltration rates prior to site disturbance; therefore, the disc permeameter method will not be discussed further in this Appendix.

Single- and double-ring infiltrometers directly measure vertical flow into the surface of the soil. Double-ring infiltrometers account for lateral flow boundary affects with the addition of an outer water reservoir and are generally the preferred method for measuring surface infiltration. Borehole permeameters are best suited to collect infiltration measurements below the soil surface, and are generally recommended for BMPs that may be installed to a deeper depth. Two such subsurface infiltration methods are discussed below, including the Guelph and falling-head permeameters.

Double-Ring Infiltrometer

The double ring infiltrometer method consists of driving two cylinders, one inside the other, into the ground, partially filling them with water, and maintaining the liquid at a constant level (ASTM D3385). The volume of water added to the inner ring (from a separate water reservoir) to maintain the constant head level is comparable to the volume of water infiltrating into the soil. The volume of water added to the inner ring divided by the time period over which the water was added and then divided by the cross-sectional area of the inner ring is equal to the infiltration rate. An image of a common double-ring infiltrometer is provided in Figure D-2.

Figure D-2: Double Ring Infiltrometer

*Photo Credit: Geosyntec Consultants (Braga and Fitsik, 2008)*

Borehole Guelph Infiltration Test

For shallow boreholes (less than 2.5 feet deep), the Guelph Permeameter has been developed as a field portable kit. This permeameter consists of a tube that is placed in a hand-drilled shallow borehole. Water is provided to the tube through a separate reservoir. Water loss in the reservoir is used to estimate the hydraulic conductivity of the soil, which may be used to calculate infiltration based on various standard models (Soil Moisture Equipment, 2005). A photograph of a Guelph Permeameter is provided in Figure D-3. It is important to remember that this method allows for both vertical and lateral water flow from the borehole and is unable to differentiate between the two flow directions. Because of this, if BMPs will be designed with impermeable liners on the sidewalls or if the BMPs will have large plan dimensions relative to their thickness, then this test may overestimate the total infiltration actually achieved by the BMP.

Figure D-3: Guelph Permeameter for Shallow Borehole Permeability

*Photo Credit: USDA, 2005*

Falling-Head Borehole Infiltration Test

The falling-head borehole infiltration test is commonly applied to assess infiltration at greater depths (e.g., between 5 and 25 ft). The method is generally performed according to United States Bureau of Reclamation procedure 7300-89 (USBR, 1990). The method consists of installing a well casing with slots cut into it, which is designed to release water at target depths, into a borehole. The borehole is then backfilled, pre-soak water is added to it, and filled again. The stage loss is recorded over the duration of the test. An example diagram is shown in Figure D-4.

The testing procedures are summarized as follows:

1. Remove any smeared soil surfaces to provide a natural soil interface for testing the percolation of water. Remove all loose material. The U.S. EPA recommends scratching the sides with a sharp pointed instrument. (Note: upon tester's discretion, a 2-inch layer of coarse sand or fine gravel may be placed to protect the bottom from scouring and sediment.) Fill casing with clean water and allow to pre-soak for 24 hours or until the water has completely infiltrated.
2. Refill casing and monitor water level (distance from top of casing to top of water) for 1 hour. Repeat this procedure a total of four times. (Note: upon tester's discretion, the final field rate may either be the average of the four observations or the value of the last observation. The final rate shall be reported in inches per hour.)
3. Upon completion of the testing, the casing may be backfilled with bentonite chips or grout if the well is not needed for future measurements.

This method allows for lateral water flow and potential vertical water flow, depending on whether the bottom of the borehole is plugged with a bentonite chip seal or not. Consequently, this test may overestimate the total infiltration actually achieved by the BMP, particularly if the BMP is designed to have predominantly vertical infiltration.

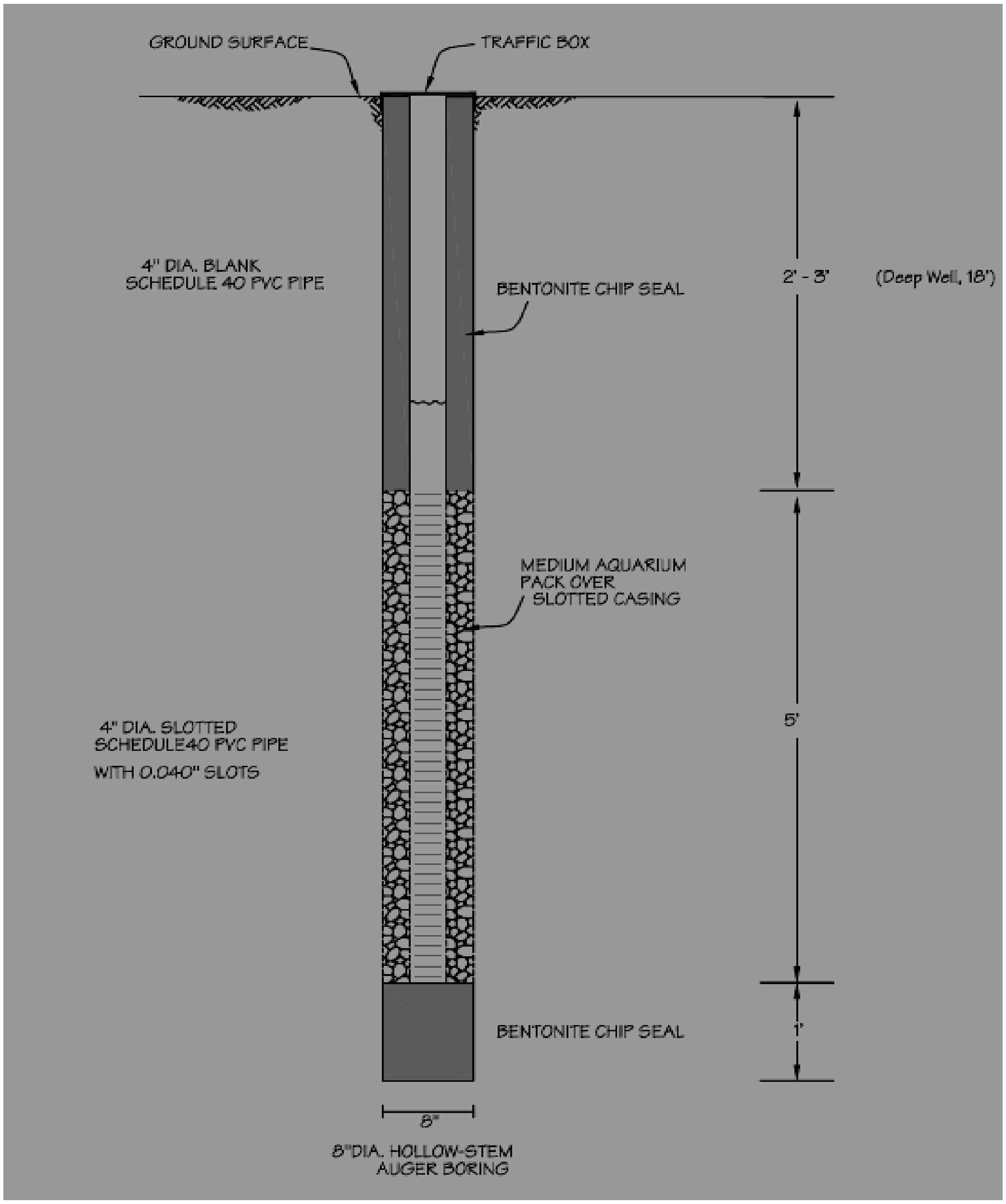



Figure D-4: Falling-Head Permeameter for Deep Borehole Permeability

*Diagram Credit: Group Delta Consultants, 2008*

<u>Laboratory Soil Tests</u>

If fill materials imported from off site are part of an infiltration BMP design or if an engineered media mix requires determination of the flow-through treatment rate, laboratory testing is required. The soil/media sample must be submitted to a certified testing laboratory. The sample must be compacted to the same degree that will be present after final grading or placement and then subjected to a hydraulic conductivity test. Laboratory methods for measuring hydraulic conductivity are generally classified as either a constant-head test or a falling-head test. The constant-head permeameter test is one of the most commonly used methods for determining the saturated hydraulic conductivity of coarse-grained soils in the laboratory (ASTM D2434 and ASTM D5084). Additional laboratory testing, including particle size analyses (ASTM D422) and Atterberg limits testing (ASTM D4318), should be performed on the fill materials to appropriately classify the soil samples. Similar laboratory tests should be performed on the proposed infiltration BMP material, which may also require organic content testing (ASTM D2974).

Laboratory testing similar to that proposed for the fill materials should also be considered in evaluating the samples of the in situ soils and may aide in reducing the required number of field infiltration tests by correlating the measured field infiltration rates with the laboratory test results of the soil.

Assessment of Test Results

The results from field infiltration methods should be examined to consider data variability and sample distribution to determine if there has been adequate sampling. If the spatial variability (heterogeneity) is large, then additional field and/or laboratory measurements and testing may be necessary. The infiltration results should be compared to the information gathered for the in situ soils and geology to see if they are consistent. The results of the in situ soils and infiltration testing may then be used in the siting, selection, sizing, and design of LID site design techniques and structural treatment BMPs.

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# APPENDIX E
# OUTLET STRUCTURE DESIGN GUIDANCE

## INTRODUCTION

Outlet structures provide the critical functions of regulating flow rates and maintaining water levels for structural storm water BMPs. Typical outlet structures for storm water facilities include:

- Orifice
- Perforated riser
- Perforated underdrain
- Pipe/culvert
- Combination outlet
- Weir
  - Broad-crested
  - Proportional
  - V-notch
  - Sharp-crested

This appendix provides design guidance for two of these outlet types that are commonly used for water quality basins: perforated risers and multi-stage orifices. The design engineer should refer to an appropriate hydraulics text for design information on the other outlet structures.

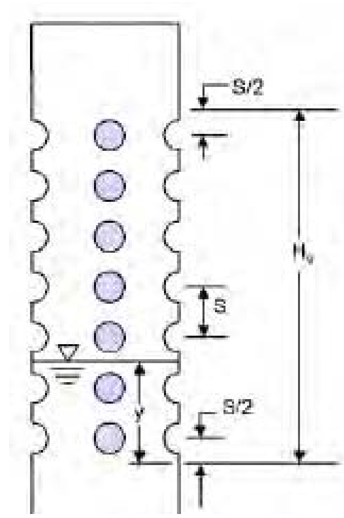
## PERFORATED RISERS OUTLET SIZING METHODOLOGY

The following attributes influence the perforated riser outlet sizing calculations**:**

- Shape of the pond (e.g., prismatic);
- Depth and volume of the pond;
- Elevation/depth of first row of perforations;
- Elevation/depth of last row of perforations;
- Size of perforations;
- Number of rows or perforations and number of perforations per row; and
- Desired draw down time (e.g., 16 hour and 32 hour draw down for top half and bottom half respectively, 48 hour total draw down time).

The governing rate of discharge from a perforated riser structure can be calculated using the equation (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s}\sqrt{2g}Y^{3/2}$$




Where:

Q = riser flow discharge (cfs)

$C_p$ = discharge coefficient for perforations (use 0.61)

$A_p$ = cross-sectional area of all the holes ($ft^2$)

s = center to center vertical spacing between perforations (ft)

$H_s$ = distance from s/2 below the lowest row of holes to s/2 above the top row of holes (ft)

Y = the distance from the bottom of the lowest hole to the water quality surface elevation (ft)

g = gravitational constant (32.2 $ft/s^2$)

For the iterative computations needed to size the perforations and determine the riser height, the above equation has been divided into two parts:

$$Q = kY^{3/2}$$

Where:

$$k = C_p \frac{2A_p}{3H_s}\sqrt{2g}$$

Uniformly perforated riser designs are defined by the depth or elevation of the first row of perforations, the length of the perforated section of pipe, and the size or diameter of each perforation. The steps needed to size a perforated riser outlet are outlined below.

Step 1: Determine riser elevation or depth in the pond

Set the riser elevation above the pond bottom to provide for sediment storage. Select a riser height such that the last row of perforations is in line with the top of the water quality pool elevation.

Step 2: Determine stage storage-area of pond

The stage storage-area curve of the pond should be determined using simple geometry, planimeter, or CAD software. The surface area should be determined for every 4-6 inches of the pond.

Step 3: Determine constant *k*

- Determine the value of the constant *k* included in the equations above that provides the desired draw down time.

- Set up a computation table in a spreadsheet program in a format such as shown in Table E-1.
- Using the pond depth, partition the pond into equal height horizontal slices to be stored as entries in Table E-1. At each elevation $E_n$ (or table entry), complete the following:
    - Determine the change in elevation $H_n$ (ft) [$H_n =( E_o – E_{n+1})$]
    - Calculate the average discharge $Q_n$ (cfs) [$Q_n = k(H_n)^{3/2}$]
    - Calculate the pond surface area $A_n$ ($ft^2$) [$A_n =$ L x W for rectangular ponds]
    - Compute the available storage $V_n$ ($ft^3$) [$V_n = A_n \times H_n$]
    - Determine the average drain time $T_n$ (hrs) [$T_n = (V_n / Q_n) \times 3600$]
- Sum up the drain times at each stage to determine the total drain time for the pond. If the value obtained is smaller or greater than the required drawdown time, increase or decrease the k value and repeat the computations above until the desired drain time is achieved.

Step 4: Determine the size and number of rows of perforations

Determine the size and number of rows of perforations that yield a *k* value equal to the *k* value used in the previous step. Follow the steps below to obtain riser attributes:

- Select an initial number of rows, number or holes per row and an initial hole diameter.
- Obtain flow area per row values from Table E-2 or compute total flow area.
- Select a value for $H_s$ and $C_p$ and compute *k*.
- Repeat the above steps varying the number of rows, hole diameter, number of holes per row and $H_s$ until the desired value of *k* is obtained or it is determined that *k* is too small to be matched by any realistic combination of inputs. Hole diameter should not be less than 1/4" to minimize the potential for clogging.

Step 5: Verify the design

The design is completed by verifying that the drain time for the top half and the bottom half are acceptable and the total drain time is equivalent to the desired value. Note that the drain time for the top half can be obtained by summing the drain times for the top half of the entries in the spreadsheet table. The drain time for the bottom half can be similarly obtained. To achieve the desired drain time for the top half, it may be necessary to first compute the required area of perforation for the top half and then compute the required area for the combination of the top half and bottom half. The area for the bottom half is then computed by subtracting the combined area from the top half area.

Table E-1: Example Spreadsheet for Perforated Riser Outlet Sizing Calculations

| LINE NO. | ELEV. (FT) | CHANGE IN ELEVATION (FT) | AVERAGE DISCHARGE (CFS) | *POND SURFACE AREA (SQ. FT) | STORAGE VOLUME (CU. FT) | AVERAGE DRAIN TIME (HRS) |
|---|---|---|---|---|---|---|
| n | $E_n$ | $H_n = (E_n - E_{n+1})$ | $Q_n = k(H_n)^{3/2}$ | $A_n$ | $V_n = A_n \times H_n$ | $T_n = V_n / Q_n$ |
| 1 | 6 | 0.3 | 0.4102 | 4256 | 1419 | 1.0 |
| 2 | 5.7 | 0.3 | 0.3765 | 3996 | 1332 | 1.0 |
| 3 | 5.3 | 0.3 | 0.3438 | 3744 | 1248 | 1.0 |
| 4 | 5.0 | 0.3 | 0.3120 | 3500 | 1167 | 1.0 |
| 5 | 4.7 | 0.3 | 0.2814 | 3264 | 1088 | 1.1 |
| 6 | 4.3 | 0.3 | 0.2518 | 3036 | 1012 | 1.1 |
| 7 | 4.0 | 0.3 | 0.2233 | 2816 | 939 | 1.2 |
| 8 | 3.7 | 0.3 | 0.1960 | 2604 | 868 | 1.2 |
| 9 | 3.3 | 0.3 | 0.1699 | 2400 | 800 | 1.3 |
| 10 | 3.0 | 0.3 | 0.1450 | 2204 | 735 | 1.4 |
| 11 | 2.7 | 0.3 | 0.1215 | 2016 | 672 | 1.5 |
| 12 | 2.3 | 0.3 | 0.0995 | 1836 | 612 | 1.7 |
| 13 | 2.0 | 0.3 | 0.0789 | 1664 | 555 | 2.0 |
| 14 | 1.7 | 0.3 | 0.0601 | 1500 | 500 | 2.3 |
| 15 | 1.3 | 0.3 | 0.0430 | 1344 | 448 | 2.9 |
| 16 | 1.0 | 0.3 | 0.0279 | 1196 | 399 | 4.0 |
| 17 | 0.7 | 0.3 | 0.0152 | 1056 | 352 | 6.4 |
| 18 | 0.3 | 0.3 | 0.0054 | 924 | 308 | 15.9 |
| 19 | 0.0 | 0.0 | 0.0000 | 800 | 0 | 0.0 |
| | | | | | Total Draw Down Time (hrs) | 48 |

* Pond surface area can be calculated or measured. Non rectangular cross sections must use the appropriate formulas for calculating cross-sectional areas.

Table E-2: Circular Perforation Sizing for Perforated Riser.

| HOLE DIA. (IN) | HOLE DIA. (IN) | MIN S (IN) | AREA PER ROW (SQ. IN) | | |
|---|---|---|---|---|---|
| | | | N=1 | N=2 | N=3 |
| 1/4 | 0.25 | 1 | 0.05 | 0.1 | 0.15 |
| 5/16 | 0.313 | 2 | 0.08 | 0.16 | 0.24 |
| 3/8 | 0.375 | 2 | 0.11 | 0.22 | 0.33 |
| 7/16 | 0.438 | 2 | 0.15 | 0.3 | 0.45 |
| 1/2 | 0.5 | 2 | 0.2 | 0.4 | 0.6 |
| 9/16 | 0.563 | 3 | 0.25 | 0.5 | 0.75 |
| 5/8 | 0.625 | 3 | 0.31 | 0.62 | 0.93 |
| 11/16 | 0.688 | 3 | 0.37 | 0.74 | 1.11 |
| 3/4 | 0.75 | 3 | 0.44 | 0.88 | 1.32 |
| 13/16 | 0.813 | 3 | 0.52 | 1.04 | 1.56 |
| 7/8 | 0.875 | 3 | 0.6 | 1.2 | 1.8 |
| 15/16 | 0.938 | 3 | 0.69 | 1.38 | 2.07 |
| 1 | 1 | 4 | 0.79 | 1.58 | 2.37 |
| 1 1/16 | 1.063 | 4 | 0.89 | 1.78 | 2.67 |
| 1 1/8 | 1.125 | 4 | 0.99 | 1.98 | 2.97 |
| 1 3/16 | 1.188 | 4 | 1.11 | 2.22 | 3.33 |
| 1 1/4 | 1.25 | 4 | 1.23 | 2.46 | 3.69 |
| 1 5/16 | 1.313 | 4 | 1.35 | 2.7 | 4.05 |
| 1 3/8 | 1.375 | 4 | 1.48 | 2.96 | 4.44 |
| 1 7/16 | 1.438 | 4 | 1.62 | 3.24 | 4.86 |
| 1 1/2 | 1.5 | 4 | 1.77 | 3.54 | 5.31 |
| 1 9/16 | 1.563 | 4 | 1.92 | 3.84 | 5.76 |
| 1 5/8 | 1.625 | 4 | 2.07 | 4.14 | 6.21 |
| 1 11/16 | 1.688 | 4 | 2.24 | 4.48 | 6.72 |
| 1 3/4 | 1.75 | 4 | 2.41 | 4.82 | 7.23 |
| 1 13/16 | 1.813 | 4 | 2.58 | 5.16 | 7.74 |
| 1 7/8 | 1.875 | 4 | 2.76 | 5.52 | 8.28 |
| 1 15/16 | 1.938 | 4 | 2.95 | 5.9 | 8.85 |
| 2 | 2 | 4 | 3.14 | 6.28 | 9.42 |

Source: Urban Drainage and Flood Control District (2005). *Drainage Criteria Manual, Volume 3, Best Management Practices.* http://www.udfcd.org/downloads/down_critmanual.htm

## MULTIPLE ORIFICE OUTLET SIZING METHODOLOGY

The following attributes influence multiple orifice outlet sizing calculations:

- The shape of the pond (e.g., trapezoidal);
- The depth and volume of the pond;
- The elevation of each orifice; and
- Desired draw-down time (e.g., 12 hour and 36 hour draw down times for top half and bottom half respectively, 48 hour draw down time for whole pond).

The rate of discharge from a single orifice can be calculated using the equation below:

$$Q = CA(2gH)^{0.5}$$

Where:

Q = orifice flow discharge (cfs)

C = discharge coefficient (unitless)

A = cross-sectional area of orifice or pipe ($ft^2$)

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

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

Multiple orifice designs are defined by the depth (or elevation) and the size (or diameter) of each orifice. The steps needed to size a dual orifice outlet are outlined below; multiple orifices may be provided and sized using a similar approach.

Step 1: Determine orifice elevations

- For the bottom orifice, set the orifice elevation ($H_b$) at a maximum of 6” above the pond bottom. If the bottom orifice is below the invert of the outlet pipe, then use the outlet pipe invert elevation for orifice calculations.
- For the top orifice, set the orifice elevation ($H_t$) at half way to the top of the water quality pool.

Step 2: Determine pond and orifice attributes and constants for computations

Parameters examined at this step include pond geometry such as pond shape, pond bottom length and bottom width and pond side slopes.

Step 3: Determine the required size of the bottom orifice

To determine the required size of the bottom orifice, follow the sizing steps below:

- Set up a computation table such as Table E-3.
- Using the pond depth, partition the pond into equal height stages to be stored as entries in Table E-4. At each elevation $E_n$ (or table entry), complete the following:

- Determine the change in elevation $H_n$ (ft) — $[H_n = (E_o - E_{n+1})]$
- Calculate the average discharge $Q_n$ (cfs) — $[Q_n = CA(2gH_n)^{0.5}]$
- Calculate the pond surface area $A_n$ ($ft^2$) — $[A_n =$ L x W for rectangular ponds$]$
- Compute the available storage $V_n$ ($ft^3$). — $[V_n = A_n \times H_n]$
- Determine the average drain time $T_n$ (hrs) — $[T_n = (V_n / Q_n) \times 3600]$

- Sum up the drain times at each stage to determine the total drain time for the bottom half of the pond. If the value obtained is smaller or greater than the desired value, increase or decrease the orifice diameter and repeat the computations in the step above until the desired drain time is achieved

Table E-3: Sample Spreadsheet for Dual Orifice Pond Outlet Sizing Calculations: Bottom Half of Pond

| LINE NUMBER | ELEVATION [E] | CHANGE IN HEIGHT | AVERAGE FLOW AT ELEV. (TOP ORIFICE ONLY) | POND SURFACE AREA* | STORAGE VOLUME | TIME TO DRAIN UNIT AT CURRENT FLOW RATE |
|---|---|---|---|---|---|---|
| | | $[E_1-E_2]$ | [See Eqn 1] | $A_{elev}$ | $[A_{elev} \times d_H]$ | $[V_{elev} / Q_{elev}]$ |
| | (ft) | H (ft) | $q_{top}$ (cfs) | ($ft^2$) | $V_{elev}$ ($ft^3$) | T (hrs) |
| 1 | 3.0 | 3.0 | 0.0567 | 2204 | 735 | 3.6 |
| 2 | 2.7 | 2.7 | 0.0534 | 2016 | 672 | 3.5 |
| 3 | 2.3 | 2.3 | 0.0500 | 1836 | 612 | 3.4 |
| 4 | 2.0 | 2.0 | 0.0463 | 1664 | 555 | 3.3 |
| 5 | 1.7 | 1.7 | 0.0422 | 1500 | 500 | 3.3 |
| 6 | 1.3 | 1.3 | 0.0378 | 1344 | 448 | 3.3 |
| 7 | 1.0 | 1.0 | 0.0327 | 1196 | 399 | 3.4 |
| 8 | 0.7 | 0.7 | 0.0267 | 1056 | 352 | 3.7 |
| 9 | 0.3 | 0.3 | 0.0189 | 924 | 308 | 4.5 |
| 10 | 0.0 | 0.0 | 0.0000 | 800 | 0 | 0.0 |
| | | | | | Subtotal Draw Down Time | 32.0 |

* Pond surface area can be calculated or measured. Non rectangular cross sections must use the appropriate formulas for calculating cross-sectional areas.

<u>Step 4: Determine the required size of the top orifice</u>

To determine the required size of the top orifice, follow the sizing steps below:

- Set up a Table such as Table E-4.
- At each elevation $E_n$ complete the following:
  - Determine the change in elevation $H_n$ (ft) — $[H_n = (E_n - E_{n+1})]$
  - Calculate the average discharge $Q_n$ (cfs) — $[Q_n = CA(2gH_n)^{0.5}]$
  - Calculate the combined average discharge $Q_0$ — $[Q_0 = q_n + q_{add}]$
  - Calculate the pond surface area $A_n$ ($ft^2$) — $[A_n =$ L x W for rectangular ponds$]$
  - Compute the available storage $V_n$ ($ft^3$) — $[V_n = A_n \times H_n]$
  - Determine the average drain time $T_n$ (hrs) — $[T_n = V_n / Q_n]$

- Note that $q_{add}$ is the maximum discharge from the bottom orifice.

- Sum up the drain times at each stage to determine the total drain time for the top half of the pond. If the value obtained is smaller than the desired value, increase or decrease the orifice diameter and repeat the computations above until the desired drain time is achieved.

Table E-4: Sample Spreadsheet for Dual Orifice Pond Outlet Sizing Calculations: Top Half of Pond

| LINE NO. | ELEVATION | CHANGE IN HEIGHT | AVERAGE FLOW AT ELEV. (TOP ORIFICE ONLY) | COMBINED AVERAGE DISCHARGE | POND SURFACE AREA | STORAGE VOLUME | TIME TO DRAIN UNIT AT CURRENT FLOW |
|---|---|---|---|---|---|---|---|
| | [E] | [$E_1$ - $E_2$] | [See Eqn 1] | [$q_{top}$ + $q_{bot}$] | $A_{elev}$ | [$A_{elev}$ x $d_H$] | [$V_{elev}$ / $Q_{elev}$] |
| | (ft) | H (ft) | $q_{top}$ (cfs) | $Q_{elev}$ (cfs) | (ft$^2$) | $V_{elev}$ (ft3) | T (hrs) |
| 1 | 6.0 | 3.0 | 0.1615 | 0.2181 | 4256 | 1419 | 1.8 |
| 2 | 5.7 | 2.7 | 0.1522 | 0.2089 | 3996 | 1332 | 1.8 |
| 3 | 5.3 | 2.3 | 0.1424 | 0.1990 | 3744 | 1248 | 1.7 |
| 4 | 5.0 | 2.0 | 0.1318 | 0.1885 | 3500 | 1167 | 1.7 |
| 5 | 4.7 | 1.7 | 0.1203 | 0.1770 | 3264 | 1088 | 1.7 |
| 6 | 4.3 | 1.3 | 0.1076 | 0.1643 | 3036 | 1012 | 1.7 |
| 7 | 4.0 | 1.0 | 0.0932 | 0.1499 | 2816 | 939 | 1.7 |
| 8 | 3.7 | 0.7 | 0.0761 | 0.1328 | 2604 | 868 | 1.8 |
| 9 | 3.3 | 0.3 | 0.0538 | 0.1105 | 2400 | 800 | 2.0 |
| 10 | 3.0 | 0.0 | 0.0000 | 0.0567 | 2204 | 0 | 0.0 |
| | | | | | | Subtotal Draw Down Time | 16.0 |
| | | | | | | Total Draw Down Time | 48.0 |

Step 5: Verify the design

The design is completed by verifying that the sum of the detention times for the top half of the pond and the bottom half of the pond add up to the total desired detention time (36 to 48 hours).

## REFERENCES

McEnroe, B.M., J.M. Steichen and R. M. Schweiger, 1988. "Hydraulics of Perforated Riser Inlets for Underground Outlet Terraces." *Trans ASAE*, Vol. 31, No. 4, 1988.

Urban Drainage and Flood Control District (2005). *Drainage Criteria Manual, Volume 3, Best Management Practices.* http://www.udfcd.org/downloads/down_critmanual.htm

# APPENDIX F
## EXAMPLE HYDRAULIC CONTROL STRUCTURE SCHEMATICS

Included in this appendix are example hydraulic control structures that can be used in BMP design. Many different outlet structure designs are possible and permissible provided they safely convey the design flow rates. The following example schematics are provided in this appendix:

1. Perforated Riser w/ Trash Screen and Low Flow Drain
2. Perforated Riser in Manhole
3. Orifice in Manhole
4. Inverted Pipe Outlet
5. Flow Spreaders and Check Dams

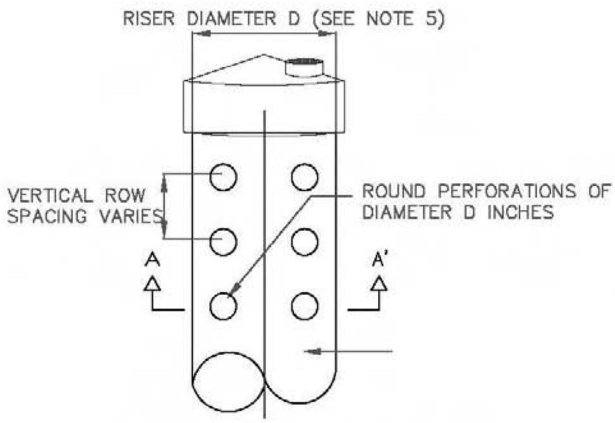
Details for additional hydraulic structures may be found at:

Urban Storm Drainage Criteria Manual Volume 3.
http://www.udfcd.org/downloads/down_critmanual_volIII.htm

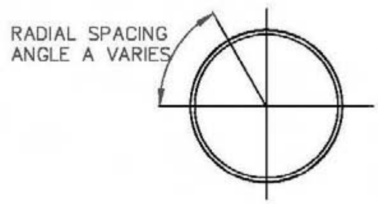
Georgia Stormwater Manual, Section 2.3 – Outlet Structures.
http://www.georgiastormwater.com/vol2/2-3.pdf

Portland Stormwater Management Manual, Appendix G – Typical Details.
http://www.portlandonline.com/bes/index.cfm?c=47963

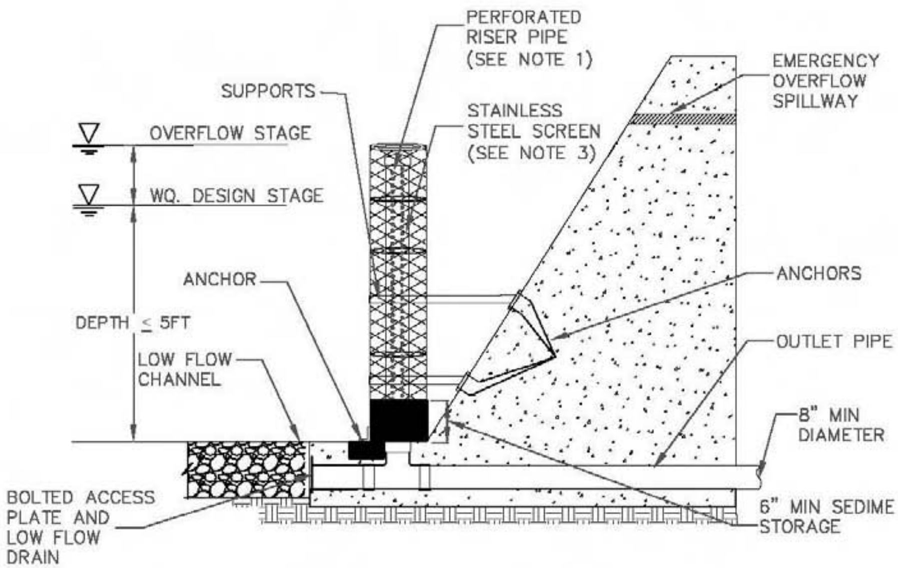
# 1. PERFORATED RISER W/ TRASH SCREEN AND LOW FLOW DRAIN




Perforated Riser Detail
(Not to Scale)




Section A – A'
(Not to Scale)




Perforated Riser Outlet Structure
(Not to Scale)

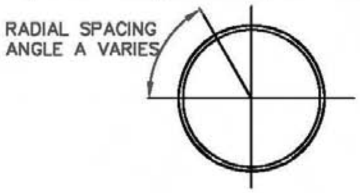
NOTES:

1. RISER PIPE SHALL BE SIZED TO PROVIDE 36 TO 48–HOUR FULL BRIM DRAW DOWN TIME.
2. TOTAL OUTLET CAPACITY: 100–YEAR PEAK FLOW FOR ON–LINE BASINS AND WATER QUALITY DESIGN FLOW FOR OFF–LINE BASINS.
3. SCREEN OPENINGS SHALL BE AT LEAST ¼" AND SHALL NOT EXCEED THE DIAMETER OF THE PERFORATIONS ON THE RISER.
4. RISER PIPE PERFORATION DIAMETER SHALL BE NO LESS THAN ½" AND NO MORE THAN 2"
5. MINIMUM PIPE DIAMETER (D) IS 2'
6. RISER PIPE MATERIAL IS CMP

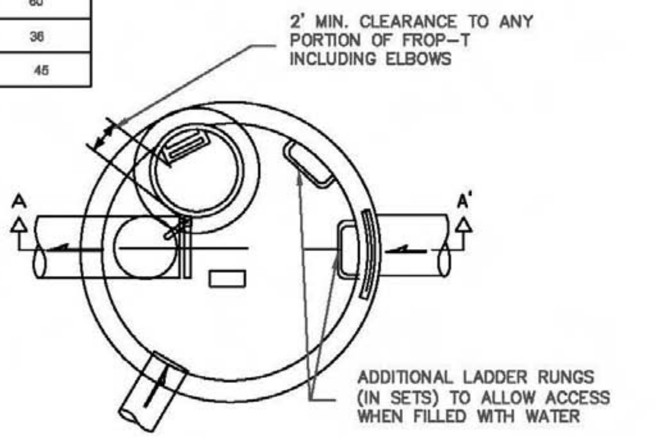
## 2. PERFORATED RISER IN MANHOLE

Smooth Plastic Riser Standard Dimensions (Wyoming NRCS)

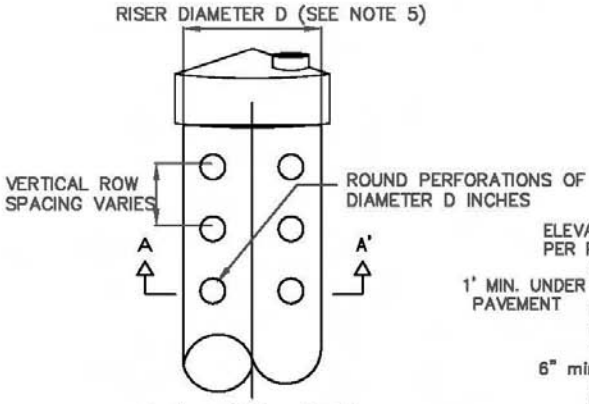
| Max Orifice Diameter D (inches) | Pipe Diameter (inches) | Min Wall Thickness (inches) | No. of vertical rows of perforations | Radial Spacing of Vertical Rows A (Degrees) |
|---|---|---|---|---|
| $3\frac{1}{2}$ | 8 | 0.15 | 6 | 60 |
| $5\frac{1}{2}$ | 10 | 0.20 | 8 | 36 |
| 6 | 12 | 0.25 | 10 | 45 |



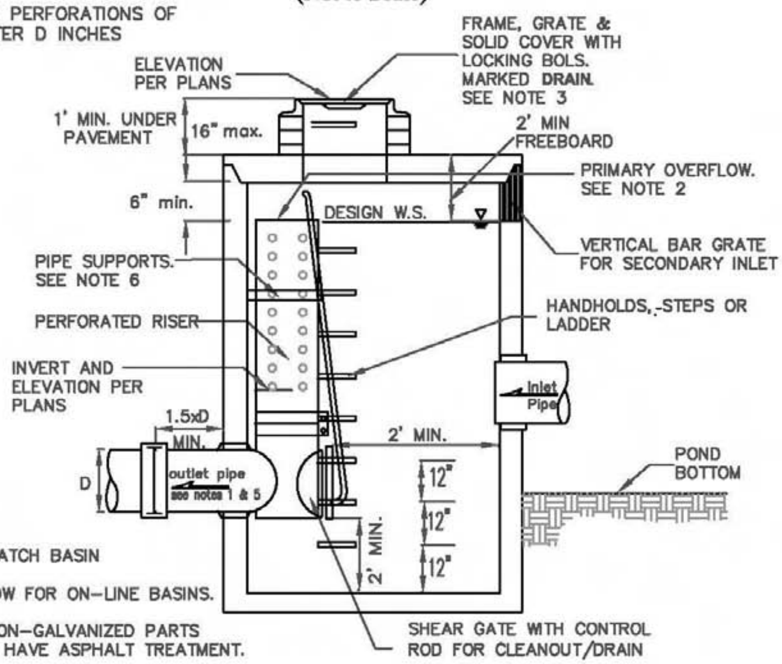

Section B-B'
(Not to Scale)




Plan View
(Not to Scale)




Perforated Riser Detail
(Not to Scale)

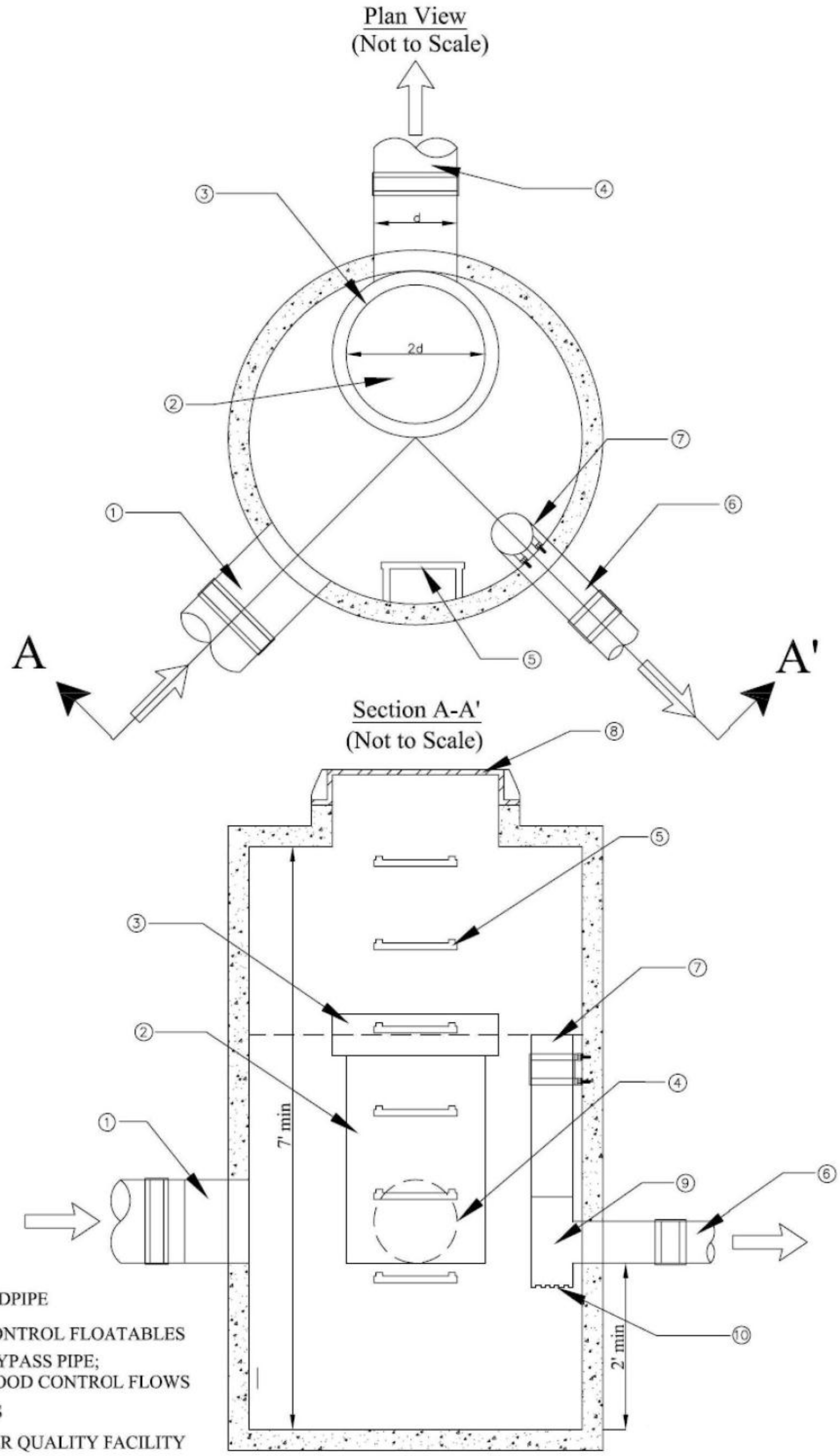



NOTES:

1. USE A MINIMUM OF A 54" DIA TYPE 2 CATCH BASIN
2. OUTLET CAPACITY: 100-YEAR PEAK FLOW FOR ON-LINE BASINS.
3. METAL PARTS: CORROSION RESISTANT. NON-GALVANIZED PARTS PERFERRED. GALVANIZED PIPE PARTS TO HAVE ASPHALT TREATMENT.
4. FRAME AND LADDER OR STEPS OFFSET SO:
   A. CLEANOUT GATE IS VISIBLE FROM TOP.
   B. CLIMB-DOWN SPACE IS CLEAR OF RISER AND
   C. FRAME IS CLEAR OF CURB.
5. IF METAL OUTLET PIPE CONNECTS TO CEMENT CONCRETE PIPE: OUTLET PIPE TO HAVE SMOOTH O.D. EQUAL TO CONCRETE PIPE I.D. LESS ¼"
6. PROVIDE AT LEAST ONE 3 X .090 GAGE SUPPORT BRACKET ANCHORED TO CONCRETE WALL. (MAXIMUM 3' VERTICAL SPACING)
7. LOCATE ADDITIONAL LADDER RUNGS IN STRUCTURES USED AS ACCESS TO TANKS OR VAULTS TO ALLOW ACCESS WHEN CATCH BASIN IS FILLED WITH WATER

Section A - A'
(Not to Scale)

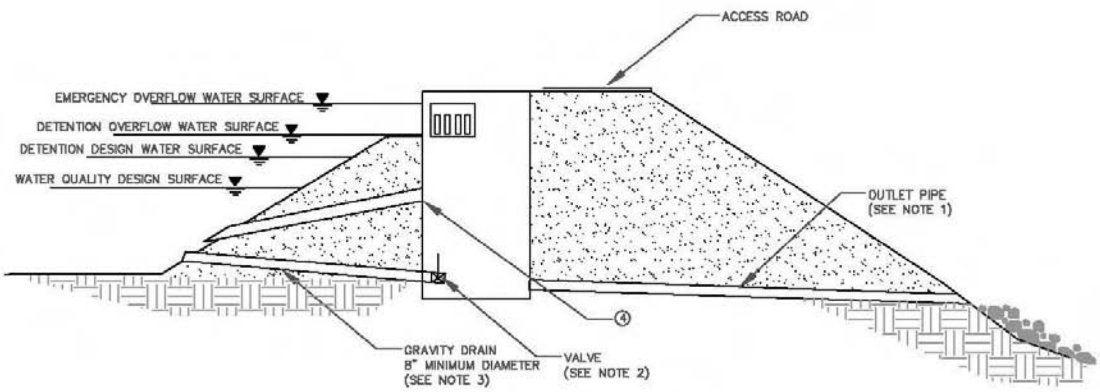
## 3. ORIFICE IN MANHOLE




NOTES:

1. INLET PIPE
2. BYPASS STANDPIPE
3. BAFFLE TO CONTROL FLOATABLES
4. HIGH FLOW BYPASS PIPE; SIZED FOR FLOOD CONTROL FLOWS
5. ACCESS STEPS
6. PIPE TO WATER QUALITY FACILITY
7. RISER PIPE; TOP OF PIPE AT DESIGN ELEVATION FOR WATER QUALITY FLOWS
8. 24" ROUND FRAME AND SOLID LID
9. REMOVABLE "TEE" SECTION FOR CLEANOUT
10. RISER PIPE ORIFICE SIZED FOR WATER QUALITY FLOWS

## 4. INVERTED PIPE OUTLET

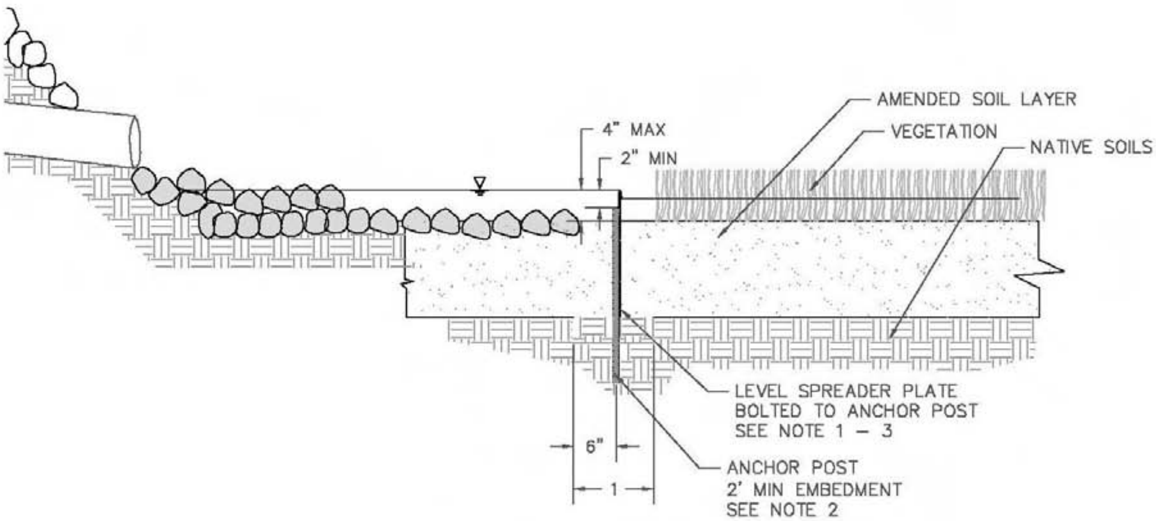



Inverted Pipe Outlet Structure
(Not to Scale)

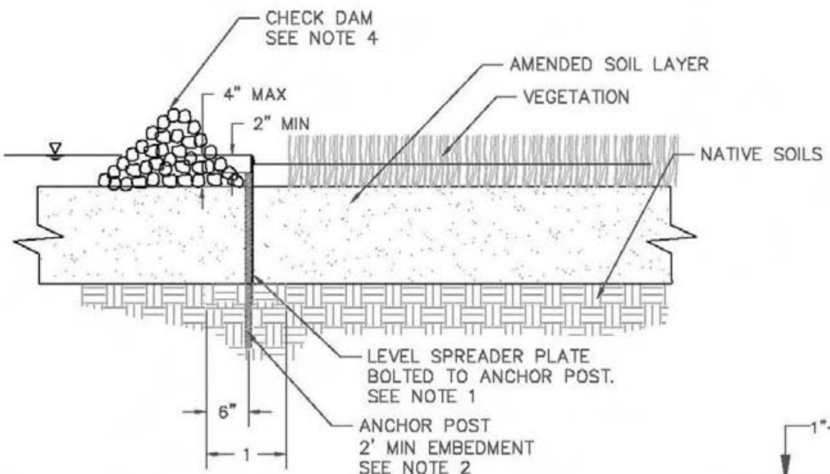
NOTES:

1. SIZE OUTLET PIPE SYSTEM TO PASS 100–YEAR FLOW FOR ON–LINE PONDS AND WATER QUALITY PEAK FLOW FOR OFF–LINE PONDS.
2. VALVE MAY BE LOCATED INSIDE MANHOLE OR OUTSIDE WITH APPROVED OPERATIONAL ACCESS
3. INVERT OF DRAIN SHALL BE 6" MINIMUM BELOW TOP OF INTERNAL BERM. LOWER PLACEMENT IS DESIRABLE. INVERT SHALL BE 6" MINIMUM ABOVE BOTTOM OF POND.
4. OUTLET PIPE INVERT SHALL BE AT WETPOOL WATER SURFACE ELEVATION

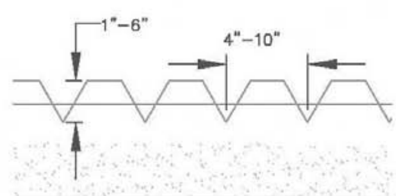
## 5. FLOW SPREADERS AND CHECK DAMS




Inlet Flow Spreader Detail
(Not to Scale)




Check Dam and Flow Spreader Detail
(Not to Scale)




V–Notched Flow Spreader Detail
(Not to Scale)

NOTES:

1. TOP SURFACE OF FLOW SPREADER SHALL BE LEVEL AND SHALL PROJECT 2" MINIMUM ABOVE GROUND. V–NOTCHES AT 6 TO 10 INCHES ON CENTER AND 1 TO 4 INCHES DEEP SHALL BE ACCEPTABLE.
2. FLOW SPREADER ANCHOR POSTS SHALL BE 4–INCH SQUARE CONCRETE , TUBULAR STEEL OR OTHER MATERIAL RESISTANT TO DECAY.
3. FLOW SPREADER PLATES SHALL HAVE A ROW OF PERFORATIONS AT THE BASE OF THE PLATE TO PREVENT PONDING FOR LONG DURATIONS.
4. CHECK DAM SHALL BE NO HIGHER THAN 12". CHECK DAM SPACING SHALL BE NO GREATER THAN 50 FEET APART.

# APPENDIX G
# BMP INSPECTION AND MAINTENANCE CHECKLISTS

## FACILITY INSPECTION AND MAINTENANCE CHECKLISTS

Included in this appendix are a series of checklists that can be used by both inspectors and maintenance personnel to ensure that observed deficiencies in BMPs are maintained appropriately. For additional maintenance information refer to the individual BMP Fact Sheets. The BMP Inspection/Maintenance Checklists are presented in the following order:

1. Bioretention/Planter Box
2. Biofiltration Swale
3. Extended Detention Basin
4. Gravity Separators
5. Media Filter
6. Permeable Pavement
7. Retention Basin/Wet Pond
8. Storm Water Wetland
9. Subsurface Vault
10. Vegetated Filter Strip

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# BIORETENTION/RAIN GARDEN AND PLANTER BOX INSPECTION AND MAINTENANCE CHECKLIST

Date: ____________________________ Work Order # ______________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ___________________________ Inspector(s): ______________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1, or 2)† | Date Maintenance Performed | Comments or Action(s) Taken to Resolve Issue |
|---|---|---|---|---|
| Appearance | Untidy | | | |
| Trash and Debris Accumulation | Trash, plant litter and dead leaves accumulated on surface. | | | |
| Vegetation | Unhealthy plants and poor appearance; when nuisance weeds and other vegetation start to take over. | | | |
| Sediment Accumulation | Sediment depth exceeds 2 inches or sediment accumulation, regardless of thickness, covers more than 10% of design area. | | | |
| Irrigation | Functioning incorrectly (if applicable). | | | |
| Inlet | Inlet pipe blocked or impeded. | | | |
| Splash Blocks | Blocks or pads correctly positioned to prevent erosion. | | | |
| Overflow | Overflow pipe blocked or broken. | | | |
| Filter media | Infiltration design rate is met (e.g., drains 36-48 hours after moderate - large storm event). | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# BIOFILTRATION SWALE INSPECTION AND MAINTENANCE CHECKLIST

Date: ___________________________ Work Order # _____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ___________________________ Inspector(s): _____________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1, or 2)† | Date Maintenance Performed | Comments or Action(s) Taken to Resolve Issue |
|---|---|---|---|---|
| Appearance | Untidy | | | |
| Trash and Debris Accumulation | Trash and debris accumulated in the swale. | | | |
| Vegetation | When the grass becomes excessively tall; when nuisance weeds and other vegetation start to take over. | | | |
| Excessive Shading | Vegetation growth is poor because sunlight does not reach swale. Evaluate vegetation suitability. | | | |
| Poor Vegetation Coverage | When vegetation is sparse or bare or eroded patches occur in more than 10% of the swale bottom. Evaluate vegetation suitability. | | | |
| Sediment Accumulation | Sediment depth exceeds 2 inches or sediment accumulation, regardless of thickness, covers more than 10% of design area. | | | |
| Standing Water | When water stands in the swale between storms and does not drain freely. | | | |
| Flow Spreader or Check Dams | Flow spreader or check dams uneven or clogged so that flows are not uniformly distributed through entire swale width. | | | |
| Inlet/Outlet | Inlet/outlet areas clogged with sediment and/or debris. | | | |
| Erosion/ Scouring | Eroded or scoured swale bottom due to flow channelization, or higher flows. Eroded or rilled side slopes. | | | |
| | Eroded or undercut inlet/outlet structures | | | |

† Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# EXTENDED DETENTION BASIN INSPECTION AND MAINTENANCE CHECKLIST

Date: ____________________________ Work Order # ______________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ___________________________ Inspector(s): ______________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1 or 2)† | Date Maintenance Performed | Comments or Action(s) Taken to Resolve Issue |
|---|---|---|---|---|
| Appearance | Untidy, un-mown (if applicable) | | | |
| Vegetation | Access problems or hazards; dead or dying trees. | | | |
| | Poisonous or nuisance vegetation or noxious weeds. | | | |
| Insects | Insects such as wasps and hornets interfere with maintenance activities. | | | |
| Rodent Holes | Any evidence of rodent holes if facility is acting as a dam or berm, or any evidence of water piping through dam or berm via rodent holes. | | | |
| Trash and Debris | Trash and debris > 5 cf/1,000 sf (one standard size garbage can). | | | |
| Pollutants | Any evidence of oil, gasoline, contaminants or other pollutants | | | |
| Inlet/Outlet Pipe | Inlet/Outlet pipe clogged with sediment and/or debris. Basin not draining. | | | |
| Erosion | Erosion of the basin's side slopes and/or scouring of the basin bottom that exceeds 2-inches, or where continued erosion is prevalent. | | | |
| Piping | Evidence of or visible water flow through basin berm. | | | |

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1 or 2)† | Date Maintenance Performed | Comments or Action(s) Taken to Resolve Issue |
|---|---|---|---|---|
| Settlement of Basin Dike/Berm | Any part of these components that has settled 4-inches or lower than the design elevation, or inspector determines dike/berm is unsound. | | | |
| Overflow Spillway | Rock is missing and/or soil is exposed at top of spillway or outside slope. | | | |
| Sediment Accumulation in Basin Bottom | Sediment accumulations in basin bottom that exceeds the depth of sediment zone plus 6-inches. | | | |
| Tree or shrub growth | Trees > 4 ft in height with potential blockage of inlet, outlet or spillway; or potential future bank stability problems. | | | |
| Tree and Large Shrub Growth on Downstream Slope of Embankments | Tree and large shrub growth on downstream slopes of embankments may prevent inspection and provide habitat for burrowing rodents. | | | |
| Inlet/Outlet Debris Barrier Damage | Debris barriers missing, damaged, or not correctly attached to pipe. | | | |
| Gate/Fence Damage | Damage to gate/fence, including missing locks and hinges | | | |

Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# GRAVITY SEPARATOR INSPECTION AND MAINTENANCE CHECKLIST

Date: ___________________________ Work Order # _____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: __________________________ Inspector(s): _____________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| **Refer to the manufacturer's instructions for maintenance/inspection requirements, below are generic guidelines to supplement manufacturer's recommendations.** | | | | |
| Sediment Accumulation in Vault | Sediment depth exceeds 6-inches in first chamber. | | | |
| Trash/Debris Accumulation | Excessive accumulation of trash and debris accumulated near inlets, outlets, or within structure. | | | |
| Sediment in Drain Pipes or Cleanouts | When drain pipes, clean-outs, become full with sediment and/or debris. | | | |
| Damaged Pipes | Any part of the pipes that are crushed or damaged due to corrosion and/or settlement. | | | |
| Access Cover Damaged/Not Working | Cover cannot be opened; one person cannot open the cover using normal lifting pressure, corrosion/deformation of cover. | | | |
| Vault Structure Includes Cracks in Wall, Bottom, Damage to Frame and/or Top Slab | Cracks wider than 1/2-inch or evidence of soil particles entering the structure through the cracks, or maintenance/inspection personnel determine that the vault is not structurally sound. | | | |
| | Cracks wider than 1/2-inch at the joint of any inlet/outlet pipe or evidence of soil particles entering through the cracks. | | | |
| Baffles | Baffles corroding, cracking warping, and/or showing signs of failure as determined by maintenance/inspection person. | | | |
| Access Ladder Damaged | Ladder is corroded or deteriorated, not functioning properly, not securely attached to structure wall, missing rungs, cracks, or misaligned. | | | |

Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# MEDIA FILTER INSPECTION AND MAINTENANCE CHECKLIST

Date: ___________________________ Work Order # _____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: __________________________ Inspector(s): _____________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Trash & Debris | Any trash and debris which exceed 5 cubic feet per 1,000 square feet of filter bed area (one standard garbage can). In general, there shall be no visual evidence of dumping.<br>If less than threshold all trash and debris will be removed as part of next scheduled maintenance. | | | |
| Inlet erosion | Visible evidence of erosion occurring near flow spreader outlets. | | | |
| Slow drain time | Standing water long after storm has passed (after 24 to 48 hours) and/or flow through the overflow pipes occurs frequently. | | | |
| Concentrated Flow | Flow spreader uneven or clogged so that flows are not uniformly distributed across the sand filter. | | | |
| Appearance of poisonous, noxious or nuisance vegetation | Excessive grass and weed growth. Noxious weeds, woody vegetation establishing, Turf growing over rock filter | | | |
| Sediment Accumulation | Sediment depth exceeds 2 inches or sediment accumulation, regardless of thickness, covers more than 10% of design area. | | | |
| Standing Water | Standing water long after storm has passed (after 24 to 48 hours), and/or flow through the overflow pipes occurs frequently. | | | |
| Tear in Filter Fabric | When there is a visible tear or rip in the filter fabric allowing water to bypass the fabric. | | | |
| Pipe Settlement | If piping has visibly settled more than 1 inch. | | | |

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Filter Media | Drawdown of water through the media takes longer than 1 hour and/or overflow occurs frequently. | | | |
| Short Circuiting | Flows do not properly enter filter cartridges. | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# PERMEABLE PAVEMENT INSPECTION AND MAINTENANCE CHECKLIST

Date: ___________________________ Work Order # _____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ___________________________ Inspector(s): _____________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Sediment Accumulation | Sediment is visible | | | |
| Missing gravel/sand fill | There are noticeable gaps in between pavers | | | |
| Weeds/mosses filling voids | Vegetation is growing in/on permeable pavement | | | |
| Trash and Debris Accumulation | Trash and debris accumulated on the permeable pavement. | | | |
| Dead or dying vegetation in adjacent landscaping | Vegetation is dead or dying leaving bare soil prone to erosion | | | |
| Surface clog | Clogging is evident by ponding on the surface | | | |
| Overflow clog | • Excessive build-up of water accompanied by observation of low flow in observation well (connected to underdrain system)<br>• If a surface overflow system is used, observation of an obvious clog | | | |
| Visual contaminants and pollution | Any visual evidence of oil, gasoline, contaminants or other pollutants. | | | |
| Erosion | Tributary area<br>• Exhibits signs of erosion<br>• Noticeably not completely stabilized | | | |
| Deterioration/ Roughening | Integrity of pavement is compromised (i.e., cracks, depressions, crumbling, etc.) | | | |

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2)† | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Subsurface Clog | Clogging is evidenced by ponding on the surface and is not remedied by addressing surface clogging. | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# RETENTION BASIN/WET POND INSPECTION AND MAINTENANCE CHECKLIST

Date: ____________________________ Work Order # ______________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ___________________________ Inspector(s): ______________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Trash & Debris | Any trash and debris which exceed 5 cubic feet per 1,000 sf of basin area (one standard garbage can) or if trash and debris is excessively clogging the outlet structure.<br><br>If less than threshold all trash and debris will be removed as part of next scheduled maintenance. | | | |
| Sediment Accumulation | Sediment accumulation in basin bottom that exceeds the depth of the design sediment zone plus 6 inches, usually in the first cell. | | | |
| Erosion | Erosion of basin's side slopes and/or scouring of basin bottom. | | | |
| Oil Sheen on Water | Prevalent and visible oil sheen. | | | |
| Noxious Pests | Visual observations or receipt of complaints of numbers of pests that would not be naturally occurring and could pose a threat to human or aquatic health. | | | |
| Water Level | First cell empty, doesn't hold water. | | | |
| Algae Mats | Algae mats over more than 20% of the water surface. | | | |
| Aesthetics | Minor vegetation removal and thinning. Mowing berms and surroundings | | | |
| Noxious Weeds | Any evidence of noxious weeds. | | | |

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Tree Growth | Tree growth does not allow maintenance access or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are not interfering, do not remove. Dead, diseased, or dying trees shall be removed. | | | |
| Settling of Berm | If settlement is apparent. Settling can be an indication of more severe problems with the berm or outlet works. A geotechnical engineer shall be consulted to determine the source of the settlement if the dike/berm is serving as a dam. | | | |
| Piping through Berm | Discernable water flow through basin berm. Ongoing erosion with potential for erosion to continue. A licensed geotechnical engineer shall be called in to inspect and evaluate condition and recommend repair of condition. | | | |
| Tree and Large Shrub Growth on Downstream Slope of Embankments | Tree and large shrub growth on downstream slopes of embankments may prevent inspection and provide habitat for burrowing rodents. | | | |
| Erosion on Spillway | Rock is missing and soil is exposed at top of spillway or outside slope. | | | |
| Gate/Fence Damage | Damage to gate/fence, including missing locks and hinges | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# STORM WATER WETLAND INSPECTION AND MAINTENANCE CHECKLIST

Date: ___________________________ Work Order # _____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: __________________________ Inspector(s): _____________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Trash & Debris | Any trash and debris which exceed 5 cubic feet per 1,000 sf of basin area (one standard garbage can). In general, there shall be no visual evidence of dumping.<br><br>If less than threshold all trash and debris will be removed as part of next scheduled maintenance. If trash and debris is observed blocking or partially blocking an outlet structure or inhibiting flows between cells, it shall be removed quickly | | | |
| Sediment Accumulation | Sediment accumulation in basin bottom at or near the depth of sediment zone. If sediment is blocking an inlet or outlet, it shall be removed. | | | |
| Erosion | Erosion of basin's side slopes and/or scouring of basin bottom. | | | |
| Oil Sheen on Water | Prevalent and visible oil sheen. | | | |
| Noxious Pests | Visual observations or receipt of complaints of numbers of pests that would not be naturally occurring and could pose a threat to human or aquatic health. | | | |
| Water Level | First cell empty, doesn't hold water. | | | |
| Aesthetics | Minor vegetation removal and thinning. Mowing berms and surroundings | | | |
| Noxious Weeds | Any evidence of noxious weeds. | | | |

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) taken to resolve issue |
|---|---|---|---|---|
| Tree Growth | Tree growth does not allow maintenance access or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are not interfering, do not remove. Dead, diseased, or dying trees shall be removed. | | | |
| Settling of Berm | If settlement is apparent. Settling can be an indication of more severe problems with the berm or outlet works. A civil engineer shall be consulted to determine the source of the settlement if the dike/berm is serving as a dam. | | | |
| Piping through Berm | Discernable water flow through basin berm. Ongoing erosion with potential for erosion to continue. A licensed geotechnical engineer shall be called in to inspect and evaluate condition and recommend repair of condition. | | | |
| Tree and Large Shrub Growth on Downstream Slope of Embankments | Tree and large shrub growth on downstream slopes of embankments may prevent inspection and provide habitat for burrowing rodents. | | | |
| Erosion on Spillway | Rock is missing and soil is exposed at top of spillway or outside slope. | | | |
| Gate/Fence Damage | Damage to gate/fence, including missing locks and hinges | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# SUBSURFACE VAULT INSPECTION AND MAINTENANCE CHECKLIST

Date: ____________________________ Work Order # ______________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ___________________________ Inspector(s): ______________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0,1, or 2) † | Date Maintenance Performed | Comments or Action(s) Taken to Resolve Issue |
|---|---|---|---|---|
| Trash & Debris | Trash and debris > 5 cf/1,000 sf (one standard size garbage can). | | | |
| Contaminants and Pollution | Any evidence of oil, gasoline, contaminants or other pollutants. | | | |
| Erosion | Undercut or eroded areas at inlet or outlet structures. | | | |
| Sediment and Debris | Accumulation of sediment, debris, and oil/grease on surface, inflow, outlet or overflow structures. | | | |
| Water drainage rate | Standing water, or by visual inspection of wells (if available), indicates design drain times are not being achieved (i.e., within 72 hours of an event). | | | |
| Apparent clogging of surface layer | Infiltrating surface caked with sediment (function may be able to be restored by replacing surface aggregate or filter cloth if provided). | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# VEGETATED FILTER STRIP INSPECTION AND MAINTENANCE CHECKLIST

Date: ___________________________ Work Order # _____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: __________________________ Inspector(s): _____________________________

| Defect | Conditions When Maintenance Is Needed | Inspection Result (0, 1 or 2)† | Date Maintenance Performed | Comments or Action(s) Taken to Resolve Issue |
|---|---|---|---|---|
| Appearance | Untidy | | | |
| Trash and Debris Accumulation | Trash and debris accumulated on the filter strip. | | | |
| Vegetation | When the grass becomes excessively tall; when nuisance weeds and other vegetation starts to take over. | | | |
| Excessive Shading | Grass growth is poor because sunlight does not reach swale. Evaluate grass species suitability. | | | |
| Poor Vegetation Coverage | When grass is sparse or bare or eroded patches occur in more than 10% of the swale bottom. Evaluate grass species suitability. | | | |
| Erosion/Scouring | Eroded or scoured areas due to flow channelization, or higher flows. | | | |
| Sediment Accumulation on Grass | Sediment depth exceeds 2 inches. | | | |
| Flow spreader | Flow spreader uneven or clogged so that flows are not uniformly distributed through entire filter width. | | | |

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

# APPENDIX H
# POST-CONSTRUCTION STORM WATER CONTROLS - MAINTENANCE AGREEMENT

## PERMIT MAINTENANCE REQUIREMENTS

As required by the Phase II Storm Water Regulations, permitees must “require all new development and redevelopment to enter into a long-term maintenance agreement and maintenance plan…” with the permittee. The agreement must allow the permittee or its designee to conduct inspections of the post-construction controls. If during the inspection process, deficiencies are found, the permittee must notify the owner or operator of the deficiencies and perform follow-up inspections to ensure the required repairs are completed. If repairs are not made, the Phase II regulations require the permittee to enforce correction orders, and if necessary, perform the needed work and assess against the owner the cost incurred for repairs. In addition, the agreement shall account for the transfer of responsibility in leases and/or deed transfers.

## SANITATION DISTRICT NO. 1

SD1 has developed a standard maintenance agreement to meet the requirements of the Phase II Storm Water Regulations. The agreement is included in this appendix of the BMP Manual and can be found on SD1’s website (http://www.sd1.org).

## CITY OF FLORENCE

The City of Florence will enforce maintenance of water quality BMP’s through it’s code board. All post-construction controls will be required to be maintained as shown in the approved development plans.

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# SD1 Post-Construction Storm Water Facility Maintenance Agreement

THIS AGREEMENT, made and entered into this ____ day of ____________, 20___, by and between (Insert Full Name of Owner)_____________________________________ hereinafter called the "Landowner", and the Sanitation District No. 1 hereinafter called SD1. WITNESSETH, that WHEREAS, the Landowner is the owner of certain real property, hereinafter called the "Property", described as:

**Group No.:** _____________

**PIDN:** _________________

The above described parcel being part of the property in Deed Book ______, Page ___ of the [County Name] County Clerks records in _____________________________________________.

WHEREAS, the Landowner is proceeding to build on and develop the Property; and

WHEREAS, the Site Plan/Subdivision Plan known as ______________________, (Name of Plan/Development) hereinafter called the "Plan", which is expressly made a part hereof, as approved or to be approved by SD1, provides for detention and water quality improvements of storm water within the confines of the Property; and

WHEREAS, SD1 and the Landowner, its successors and assigns, including any homeowners association, agree that the health, safety, and welfare of the residents of [Local Jurisdiction], Kentucky, require that on-site storm water management facilities be constructed and maintained on the Property; and

WHEREAS, SD1 requires that on-site storm water management facilities as shown on the Plan be constructed and adequately maintained by the Landowner, its successors and assigns, including any homeowners association.

NOW, THEREFORE, in consideration of the foregoing premises, the mutual covenants contained herein, and the following terms and conditions, the parties hereto agree as follows:

1. The on-site storm water management facilities shall be constructed by the Landowner, its successors and assigns, in accordance with the plans and specifications identified in the Plan. The Post-Construction Storm Water Facility Maintenance Agreement completed by the Landowner shall be submitted at the time that the site/subdivision is platted. The execution of the within Agreement by SD1 shall not constitute an approval or acceptance of the completed storm water management facilities by the Landowner, which approval, if given, shall be by a separate written document executed by SD1.

2. The Landowner, its successors and assigns, including any homeowners association, shall adequately maintain the storm water management facilities. This includes all private pipes, channels or other conveyances built as part of the facility, as well as all structures, improvements, and vegetation provided to control the quantity and quality of the storm water. Adequate maintenance is herein defined as good working condition so that the facilities are performing their design functions. The BMP Inspection and Maintenance Checklists (as found in the Storm Water Best

Management Practices Manual) are to be used to establish what good working condition is acceptable to SD1.

3. The Landowner, its successors and assigns, shall inspect the storm water management facility and complete an inspection report annually. The purpose of the inspection is to assure safe and proper functioning of the facilities. The inspection shall cover the entire facilities, berms, outlet structure, pond areas, access roads, etc. Deficiencies, proposed corrective actions and a schedule for corrective actions shall be stated in the inspection report. Inspection reports shall be maintained and be made available to SD1 upon request for review.

4. The Landowner, its successors and assigns, hereby grant permission to SD1, its authorized agents and employees, to enter upon the Property and to inspect the storm water management facilities whenever SD1 deems necessary. SD1 shall provide the Landowner, its successors and assigns, copies of its inspection findings and a directive to commence with the repairs if necessary.

5. In the event the Landowner, its successors and assigns, fails to maintain the storm water management facilities in good working condition acceptable to SD1, SD1 may enter upon the Property and make such repairs, replacements or maintenance to the storm water management facility as may be necessary in SD1's sole judgment and to charge the costs of such repairs, replacements or maintenance to the Landowner, its successors and assigns. This provision shall not be construed to allow SD1 to erect any structure of a permanent nature on the land of the Landowner outside of the easement for the storm water management facilities. It is expressly understood and agreed that SD1 is under no obligation to routinely maintain or repair said facilities, and in no event shall this Agreement be construed to impose any such obligation on SD1.

6. The Landowner, its successors and assigns, will perform the work necessary to keep the storm water management facilities in good working order as appropriate. In the event a maintenance schedule for the storm water management facilities (including sediment removal) is outlined on the approved plans, the schedule will be followed.

7. In the event SD1 pursuant to this Agreement, performs work of any nature, or expends any funds in performance of said work for labor, use of equipment, supplies, materials, and the like, the Landowner, its successors and assigns, shall reimburse SD1 upon demand, within thirty (30) days of receipt thereof for all actual costs incurred by SD1 hereunder.

8. This Agreement imposes no liability of any kind whatsoever on SD1 and the Landowner agrees to hold SD1 harmless from any liability in the event the storm water management facilities fail to operate properly.

9. This Agreement shall be recorded among the land records of [Local Jurisdiction], Kentucky, and shall constitute a covenant running with the land, and shall be binding on the Landowner, its administrators, executors, assign, heirs and any other successors in interests, including any homeowners association.

WITNESS the following signatures

______________________________________________________________
Company/Corporation/Partnership Name

By:___________________________________________________________

______________________________________________________________
(Type Name and Title)

The foregoing Agreement was acknowledged before me this _____ day of ____________,

20___, by ________________________________________________________________.

____________________________________
NOTARY PUBLIC
My Commission Expires: ______________

County of _____________, Kentucky

Sanitation District No. 1

By:___________________________________________________________

______________________________________________________________
(Type Name and Title)

The foregoing Agreement was acknowledged before me this _____ day of ____________,

20___, by ________________________________________________________________.

____________________________________
NOTARY PUBLIC
My Commission Expires:______________

County of _____________, Kentucky

By:____________________________________________________

____________________________________________________
(Type Name and Title)

The foregoing Agreement was acknowledged before me this _____ day of ____________,

20__, by ________________________________________________________.

______________________________
NOTARY PUBLIC
My Commission Expires:_____________

County of _____________, Kentucky

This Instrument Prepared By:

______________________________ ______________________
ATTORNEY DATE

____________________________________________________
NAME AND TITLE

____________________________________________________
FIRM NAME

____________________________________________________
ADDRESS

# APPENDIX I

## POST-CONSTRUCTION STORM WATER CONTROLS - INSTALLATION CERTIFICATION

As required by the Phase II Storm Water Regulations, permitees shall “develop procedures for a post-construction process to demonstrate and document that post-construction storm water measures have been installed per design specifications, which includes enforceable procedures for bringing noncompliant projects into compliance.”

SD1 has developed a standard installation certification to meet the requirements of the Phase II Storm Water Regulations. The Post-Construction BMP Installation Certification is to be completed by the contractor during the construction and installation of post-construction BMPs to ensure BMPs are constructed in accordance with approved design plans. This is a requirements of the Phase II Storm Water Regulations. The certification is included in this appendix of the BMP Manual and can be found on SD1’s website (http://www.sd1.org).

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# SD1 BMP Installation Certification

SD1 Project ID Number: ______________________________

Project Name: ______________________________

Project Address: ______________________________

Storm Water Best Management Practice(s) installed: ______________________________

Note: This certification statement must be executed by the contractor constructing the post-construction BMP. Periodic construction observation by the certifying person will be required to fulfill this certification. The certifying person must supply the design engineer, who is not required to be on-site during the construction of the BMP, with the complete Installation Certification Checklist and record drawings in order to approve and confirm design compliance, after construction is complete.

## CONTRACTOR CERTIFICATION STATEMENT

To the best of my knowledge, I hereby certify that the storm water management facilities have been installed in accordance with the approved construction drawings, design documents, specifications, and/or any approved modifications, on file with SD1 except as noted on the record drawings.

☐ Included with this certification statement from the contractor is all necessary supporting documentation as outlined on Page 2 of this document.

Contractor: ______________________________

Signature: ______________________________

Printed Name: ______________________________ Date: ____________

## ENGINEER STATEMENT OF CONFIRMATION

Based upon my review of the Installation Certification Checklist, provided by the contractor, I confirm that the constructed BMP is consistent with the intent of the approved design. Furthermore, the documented changes on the record drawings do not adversely impact the required performance or safety aspects of the facility and comply with SD1's Storm Water Rules and Regulations.

Engineer: ______________________________

Signature: ______________________________

Printed Name: ______________________________ Date: ____________

The following supporting documents are included with the hard copy and PDF of this BMP Installation Certification:

- ☐ Copy of the contractor's record drawings for the facility (PDF).
- ☐ Copy of the revised materials summary sheet for the facility based on as-built conditions (PDF). This should include copies of material delivery tickets (see Installation Certification Checklist for required materials).
- ☐ An original completed copy of the Installation Certification Checklist required by SD1 for each constructed BMP (PDF).
- ☐ Color digital photographs of the required installation components (per the Installation Certification Checklist) and the completed facility.
- ☐ Copy of the landscape company's letter certifying the installation of the specific plants required at the facility (PDF).
- ☐ Documentation by the supplier of the amended soil that the biofiltration soils mix meets the required specifications (Original Hard Copy and PDF).

Additional items included: ______________________________________________

______________________________________________________________________

______________________________________________________________________

______________________________________________________________________

______________________________________________________________________

Additional Comments: ________________________________________________

______________________________________________________________________

______________________________________________________________________

______________________________________________________________________

______________________________________________________________________

Project Name: ________________________________ BMP Type: ____________________

BMP Location on Project Site: ________________________________________________

| INSTALLATION CERTIFICATION CHECKLIST | | | | | |
|---|---|---|---|---|---|
| **Feature Component** | **Unit** | **Required** | **Actual** | **Picture ID** | **Date/Initials** |
| Diameter of Underdrain | Inches | | | | |
| Slope of Underdrain | Ft/Ft | | | | |
| Length of Underdrain | Feet | | | | |
| Depth of Gravel | Feet | | | | |
| Installation of Filter Fabric (list locations) | Yes/No | | | | |
| Type of Filter Fabric | Specification | | | | |
| Depth of Biofiltration Soil Mix | Feet | | | | |
| Placement of Mulch | Yes/No | | | | |
| Depth of Mulch | Inches | | | | |
| Plant Spacing | Spacing | | | | |
| Type and Number of Plants | Specification | | | | |
| Filter Bed Depth | Feet | | | | |
| Depth of Permeable Pavement/ Paver Thickness | Inches | | | | |
| Overflow Structure Rim Elevation | Feet | | | | |
| Overflow Structure Invert Elevation | Feet | | | | |
| Installation of Cleanout | Yes/No | | | | |
| BMP Footprint | Square Feet | | | | |
| Other:______________ | | | | | |
| Other:______________ | | | | | |

Additional Comments: ____________________________________________________

________________________________________________________________________

________________________________________________________________________

Contractor's Signature: ______________________________ Completion Date: ______________

# APPENDIX J

## LIMNOTECH TECHNICAL MEMORANDUMS ON WATERFOWL AND MOSQUITOS

The following Memorandums are included in this appendix:

1. Bradley, Doug and Scott Bell. Waterfowl and Storm Water Basins. Prepared for Sanitation District No. 1. LimnoTech. 2011.

2. Bradley, Doug and Scott Bell. Mosquitoes and Storm Water Basins. Prepared for Sanitation District No. 1. LimnoTech. 2011.

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LimnoTech
Water | Environment | Scientists | Engineers

**DATE:** May 3, 2011

# MEMORANDUM

**FROM:** Doug Bradley
Scott Bell

**PROJECT:** KYSDCP1A – Task 5.99

**TO:** Jim Gibson, Sanitation District No. 1 of Northern Kentucky (SD1)

**CC:** Project File

**SUBJECT**: Waterfowl and Storm Water Basins

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## Executive Summary

This memo summarizes a literature review to compile information on the presence and management of waterfowl in storm water basins. The key questions investigated in this review are:

- Do waterfowl contribute to bacteria in storm water basins or storm water wetlands?
- What are the recommended methods for preventing or minimizing waterfowl use in storm water basins or storm water wetlands?
- Is there any documentation on the effectiveness of exclusion methods?

Available studies that investigated existing storm water basins and examined the relationship between basin design and waterfowl production were reviewed and are summarized. Studies tended to focus on waterfowl species that are most problematic in urban areas – Canada geese and mallard ducks. Studies find a relationship between waterbird abundance and bacterial contamination in recreational waters, but studies are lacking on the impact of contaminations within retention basins and constructed wetlands. Further, the human health risk of waterbird-derived bacteria remain understudied and unknown but the increasing use of urban ponds by highly-adaptive species like Canada geese and mallard ducks continue to raise concerns about potential health risks.

Key findings and recommendations for control measure designs to deter problem waterfowl use include the following:

- Local waterfowl management measures are most effective if considered in collaboration with regional scale efforts to deter problem species use.
- Deterrent measures included in the basin design phase are more effective that those implemented at the post-constructed response phase.
- Implementing multiple waterfowl deterrent techniques are more effective than any single technique at discouraging constructed basin and wetland use by problem species.
- Tall, thick vegetation surrounding the waterline appears to be the single-most effective approach for discouraging Canada geese use, but this approach was less effective at discouraging mallard duck use.
- Frequent mowing of the perimeter of basins and wetlands for aesthetic purposes is a common practice but encourages use by these problem species.

## Introduction

The treatment and aesthetic value of the retention basins (wet ponds) have greatly expanded the number and distribution of waterfowl habitats in urban environments (Smith 2006). Unfortunately, an unintended consequence of these treatment basins is that many have developed into nuisance attractors to waterfowl species such as Canada geese and mallard ducks whose large size (geese), large numbers (geese) and volume of fecal matter (geese and ducks) have created concerns and conflicts over waterfowl impacts at these facilities (Titchenell and Lynch, 2010; Smith, 2006; Smith et al., 1999).

Titchenell and Lynch (2010) cite primary conflicts over Canada geese use in urban areas as the following:

- Feces accumulation
- Degraded water quality
- Property damage
- Human attacks (especially children)
- Vehicle damage (roadway accident related)
- Agriculture damage

Canada geese and mallard ducks are both highly adaptable waterfowl that thrive in urban environments. Canada (C.) geese populations in North America are very large and continue to grow because of their ability to use constructed water features found in urban environments (Smith et al., 1999). Urban environments greatly increase the survival rates of C. geese because of the increasing availability and access to areas that meet their reproductive and brood-rearing needs and the absence of predators and hunting pressure found in urban environments (Smith, 2006).

Mallard ducks (mallards) are ubiquitous and the most common duck species found in North America. Mallards are particularly attracted to urban settings in the winter because constructed ponds have longer ice-free periods and because year-round, human derived, supplemental feeding is common in urban settings (Smith et al., 1999). However, mallard numbers don't appear to be as much of a nuisance in urban settings as C. geese (Smith, 2006).

Both C. geese and mallards are commonly found together because of their adaptive abilities, similar reproductive requirements, and attraction to the open-water, easily accessible shoreline habitats and abundant food sources created by many constructed ponds (Smith 2006). The distribution, availability and suitability of these constructed ponds for waterfowl species like C. geese and mallards have resulted in human health concerns based on degraded water quality caused by waterbird feces containing a variety of pathogens such as Giardia and Coliform bacteria (Titchenell and Lynch, 2010). Waterbirds are considered an important nonpoint source of bacteria inputs into surface waters, but their human health concern contribution is difficult to quantify (Kirschner et al., 2004). Studies suggest that C. geese and mallard numbers and their use of constructed ponds continues to rise, increasing concerns about the potential for future human health impacts from nuisance waterfowl (Smith, 2006).

The purpose of the memo, is to 1) documented research on whether waterfowl contribute bacteria to storm water basins or wetlands, 2) recommend methods for excluding waterfowl from storm water basins or wetlands, and 3) document the effectiveness of exclusion measures.

## Bacteria from Waterfowl in Constructed Basins

The expanding use of retention basins and constructed wetlands combined with increasing populations of species such as C. geese and mallards has resulted in research examining the human health concerns and risks from waterfowl sources of bacteria in these basins and wetlands. Waterfowl contain bacteria in their intestines that are known human pathogens and these bacteria are known to contaminate waterbodies (Abulreesh et al., 2004). Smith (2006) states that despite the concerns over health risks associated with bacteria like *E. coli* from waterbirds, actual risk-related research is largely understudied. Literature summaries on bacteria-specific impacts from waterfowl on humans in recreational waters are numerous, and recent work by Kirschner et al. (2004) documented significant correlations between waterbird abundance and bacteria contaminants in recreational waters.

Unfortunately, studies focused on constructed basins are relatively few. Reviews such as Fleming and Fraser (2001) and Smith (2006) provide valuable supporting documentation on the potential bacteria impact that waterfowl can have in constructed basins. Both authors recommend further research to specifically quantify public health sources of bacteria from waterfowl and the survive time of pathogens once in the environment.

Fleming and Fraser (2001) reviewed many types of waterfowl from selected studies conducted between 1965 and 1999 and concluded that fecal-derived sources of bacteria in waterbodies vary by species, density, feeding habits, season of waterfowl use and dilution capacity of waterbody. The risk of bacteria contamination seemed to be higher in, 1) waterbodies containing dense waterfowl use, 2) dominant species with high rates of infection (C. geese, mallards), 4) species with large individuals (C. geese), 5) small waterbodies, and 6) waterbodies with high residency periods.

Smith (2006) evaluated storm water basins specifically, but focused on C. geese and mallards. C. geese appear particularly problematic because they produce large and voluminous feces per bird, and a direct relationship is found between number of infectious oocysts from pathogens and weight of fecal sample. This fecal weight to infection raises the human health risk of fecal contamination in high C. geese and mallard use areas such as storm water basins. Storm water basin use by C. geese and mallards are increasing but the human health risk remains unknown. Smith (2006) found that despite their focused research on constructed basins and waterfowl use, no conclusive evidence linked *E. coli* to waterfowl. *E. coli* levels did increase during rain events but the sources did not appear waterfowl derived.

## Waterfowl Exclusion Measures and Effectiveness

Clearly more research is needed to understand the human health risk that constructed basins serve as potential sources of waterbird-borne bacteria. As described above, Kirschner et al. (2004) documented the relationship between bird abundance and problem bacteria in recreation waters. So, lacking constructed wetland focused research, a conservative approach assumes that reductions in the attractiveness of these facilities to high-risk, bacteria transmitting species such as C. geese and mallards, will result in reduced bird numbers and associated contaminant bacterial inputs. The following is a summary of the various exclusion measures that can be applied to constructed wetland facilities with qualitative evaluations of their effectiveness.

Prior to implementing strategies in existing facilities to reduce nuisance waterfowl, it is important that facility managers recognize and implement federal and state protections that these species are categorized under:

1. All migratory birds are protected under the 1918 Migratory Bird Treaty Act. This protection provides a wide range of protections that range from general harassment to feather collection without a permit or outside a federally approved hunting season (Titchenell and Lynch, 2010).
2. All states (as of 1999) have been given management authority by U.S. Fish and Wildlife Service to administer special C. geese permit authority to manage nuisance C. geese populations (Titchenell and Lynch, 2010).
3. All states have State Wildlife Management Plans that should be consulted for direction or contacts for species specific considerations prior to implementation of any exclusion measures.

Smith et al., (1999) produced a comprehensive guide for managing C. geese in urban environments. Although the guide is directed at C. geese, many of the approaches are applicable to other waterbirds such as mallards. The guide is titled, “Managing Canada Geese in Urban Environments, a technical guide”. Many of the recommendations in Smith et al. (1999) are also included in more recent and comprehensive waterfowl management documents such as Titchenell and Lynch (2010) and Smith (2006). Finally, many of the recommendations are suggested as response treatments rather than pre-construction design, preventative measures. In situations where waterfowl species and numbers may be a concern, proactive approaches at the design phase will likely yield better results than post-construction responses (Smith et al., 1999).

Overall, waterfowl management should be conducted at the site level, but should incorporate local and regional information to better understand the potential and degree of existing and uses of problem waterfowl. A mix of waterfowl management plans and programs will improve the effectiveness at the facility of interest and should include the following (Smith et al., 1999),

General Management Strategies

1. Understanding species ecology and limits will improve management effectiveness,
2. Understanding public attitude will aid in the selection of acceptable management options, and
3. Developing an integrated strategy has been proven to be most effective because no single technique is equally best at all locations.

Techniques for reducing site attractiveness are described and summarized in Table 1. In summary, Smith et al. (1999) suggests that multiple techniques (Table 1) are required as the most effective approach for nuisance waterbird reductions. Smith (2006) states that vegetation height is the most important individual characteristic influencing C. geese at constructed wetland use, but found that vegetation height was less influential on duck presence. Doncaster and Keller (1998) provide several illustrated examples of control techniques.

## Table 1. Techniques for Reducing Basin Use by Waterfowl

| Technique | | Effectiveness | Implementation stage | | | | |
|---|---|---|---|---|---|---|---|
| Category | Subcategory | | Design | Post design | Long-term management | Permit/Consultation Required | General Comments |
| Supplemental Feeding | | Low/Moderate | | X | X | X | Public education and public buyoff needs to be acceptable and implementable to be effective. |
| Habitat Modification | | | | | | | Most effective during the planning and design phase. These need to be combined with other methods to be most effective. |
| | Shoreline design | Moderate | X | X | X | X | Eliminate or reduce straightened shorelines, islands and peninsulas. Most effective if combined with feeding bans and the addition of near-shore walking paths. |
| | Shoreline maintenance | High | X | X | X | | Ponds with grassy perimeters should be surrounded by mature grasses. Mowing is generally conducted for aesthetic purposes only. C. geese and mallards are attracted to groomed, short, newly sprouting grasses. Where mowing is needed, mow infrequently and to a height no less than 2.5 inches. |
| | Near shore walking paths | Low/Moderate | X | X | | | Near shore paths tend to discourage waterfowl use. Most effective if combined with other techniques as described above. |
| | Groomed grassy area locations | Low | X | X | X | | Category for athletic fields. Seasonally (during molting) effective if placed over 450ft from water. |
| | Removal of nesting structures | Low/Moderate | X | X | X | X | These are often well-intended installations and desirable for adaptable species like C. geese and mallards. Eliminating potential nesting structures may reduce local reproduction use of the facility. |
| | Pond level variation | Low | X | | X | | This can be effective if combined with other shoreline techniques because it may discourage nesting use. This approach needs to consider the potential to affect facility management and impacts on mosquito production. |
| | Pond freeze-up | Low | X | X | X | | Winter use by waterfowl is increased where year – round open water (unfrozen) is available. Allowing winter freezing reduces and eliminates winter use for many nuisance species. |
| | Overwater grid wires | Moderate | | X | X | | Best if combined with other techniques such as fencing. This method can be effective but requires continuous maintenance and is generally |

| Technique | | Effectiveness | Implementation stage | | | | |
|---|---|---|---|---|---|---|---|
| Category | Subcategory | | Design | Post design | Long-term management | Permit/Consultation Required | General Comments |
| | | | | | | | unattractive to people and can reduce or eliminate use by all flying birds. |
| | Fence barriers | Moderate | | X | X | | Best if combined with other techniques. Most effective during prenesting and flightless periods during early summer. Requires regular maintenance. |
| | Vegetative barriers | Moderate | X | X | X | | Best if included in initial design and most effective if combined with other design techniques. Purpose is to reduce perceived escape routes if waterfowl are threatened and remove visual connection to waterbody. Tall (> 30”), wide (~ 20’) and dense plantings are most effective at deterring waterfowl walking access. |
| | Rock barriers | Low/Moderate | X | X | | | Best if included in initial design and combined with vegetative barrier techniques. Boulders of at least 2 ft. placed along the shoreline tend to challenge waterfowl (C. geese and mallards) that like to walk out of water. |
| | Tall trees | Low | X | X | X | | Only effective at disrupting flight path access to waterbody. Most effective with small ponds where trees interrupt flight path to and from pond. Otherwise, may actually be attractive to waterfowl with easy walking access to waterbody. |
| | Decrease grazing foods | Low | X | X | X | | Effective if combined with barriers and other shoreline techniques. Geese prefer young grass shoots and fertilized, well watered lawns. Reduce or eliminate mowing near water’s edge. Reduce maintained lawn in vicinity of pond. Plant unpalatable grasses and other vegetation. |
| | Diversionary feeding areas | Low/Uncertain | | | X | | Best if applied in rural areas and when combine with other management plans for other local heavy impact areas. This method should be evaluated before implementation because its effectiveness is uncertain in most urban areas and may increase nuisance problem. |
| Hazing and Scaring | | | | | | | Nonlethal methods designed to frighten waterbirds away from problem areas. Need to be combined with other techniques and federal and local laws |

| Technique | | Effectiveness | Implementation stage | | | | |
|---|---|---|---|---|---|---|---|
| Category | Subcategory | | Design | Post design | Long-term management | Permit/Consultation Required | General Comments |
| | | | | | | | and regulations need to be considered prior to implementation. Generally impacts all waterfowl and will disperse them to other local areas. |
| | Noisemaking devices | Low | | X | X | X | May be unacceptable alternative in urban environments. Geese tend to become habituated to devices. Can be costly and high maintenance. |
| | Visual frightening devices | Low | | X | X | X | Some methods are commonly implemented in urban environments. Geese tend to become habituated to many of these devices. Costs are highly variable depending on approach. Methods include;<br>Strobelights, mylar tape, flags, eye-spot balloons or kites, scarecrows, dogs, swans, falcons, radio-controlled aircraft, vehicles and boats. |
| Chemical Repellents | | Low/Moderate | | X | X | X | Visually and acoustically unobtrusive. Expensive and high maintenance. Duration of effectiveness highly variable. Only effective for some waterfowl behaviors (grazing). May adversely impact non-target organisms. |
| Reproduction Control | | | | | X | X | The most effective way to decrease the size of an urban flock is to increase mortality. Generally these methods are best if combined with other discouraging techniques. |
| | Hunting | Low/Moderate | | | X | X | Now the major cause of adult waterfowl losses but generally not an option in urban settings. |
| | Nesting material removal | Low | | | X | X | Waterfowl tend to delay egg laying until suitable nests are available. Removing nest during breeding causes birds to relocate, build new nests, or nest later in the season. The approach is very labor intensive and costly and only effective in small, accessible, nesting areas. |
| | Oiling, addling, puncturing eggs | Low | | | X | X | Each technique is designed to kill the embryo but retain the egg in the nest to discourage additional egg production. Technique is labor intensive (costly) and should only be conducted by experience personnel. |
| | Dummy egg replacement | Low | | | X | X | This technique is designed to remove fertilized egg and replace with an unfertilized replicate and is |

| Technique | | Effectiveness | Implementation stage | | | | |
|---|---|---|---|---|---|---|---|
| Category | Subcategory | | Design | Post design | Long-term management | Permit/Consultation Required | General Comments |
| | | | | | | | meant to discourage additional egg production. Technique is labor intensive (costly) and should only be conducted by experience personnel. |
| | Sterilization | Low | | | X | X | This technique involves neutering male birds, is labor intensive, expensive and questionably effective. |
| Bird Removal | | Moderate | | | X | X | The technique involves the direct capture and removal of birds and results are obvious and immediate. However, the technique is labor intensive and expensive and requires experienced personnel and extensive coordination because it requires handling and movement of live birds. |

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**DATE:** April 26, 2011

# MEMORANDUM

**FROM:** Doug Bradley
Scott Bell

**PROJECT:** KYSDCP1A – Task 5.99

**TO:** Jim Gibson, Sanitation District No. 1 of Northern Kentucky (SD1)

**CC:** Project File

**SUBJECT**: Mosquitoes and Storm Water Basins.

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## Executive Summary

This memo summarizes a literature review to compile information on the presence and management of mosquitoes in storm water basins. The key questions investigated in this review are:

- What conditions in storm water basins or storm water wetlands lend themselves to mosquito breeding?
- What are the recommended methods for preventing mosquito breeding in storm water basins or storm water wetlands?
- Is there any documentation on the effectiveness of prevention methods?

Available studies that investigated existing storm water basins and examined the relationship between basin design and mosquito production were reviewed and are summarized. These studies generally supported the need to avoid shallow, stagnant water conditions in retention basins and constructed wetlands. Key findings and recommendations for storm water control measure design in areas where minimization of mosquito production is desired include the following:

- In general, the conditions most favorable to mosquito production are shallow (i.e., < 12” deep), stagnant waters.
- Basin and wetland design should minimize standing water less than 12” deep.
- Basins and wetlands should be designed to maximize circulation and minimize or eliminate stagnant areas.

Unfortunately, there is almost no quantitative data available in the literature to evaluate the effectiveness of these techniques on limiting mosquito production.

## Introduction

Storm water basins and wetlands are common control measures for storm water management nationally and in Northern Kentucky. Both of these control measure types include a permanent pool of water and a commonly raised question is whether the permanent pool provides breeding habitat for mosquitoes. Concerns about these basins and wetlands elsewhere have been that these facilities provide the unintended consequence of creating havens for mosquito breeding by increasing the availability of breeding locations, lengthening mosquito producing seasons, and producing greater numbers of mosquitoes relative natural production areas (Harbison et al., 2010). Further, the greater public health concern is that these treatment designs may provide breeding grounds for mosquitoes that carry diseases such as malaria, yellow fever, and West Nile virus (Harbison and Metzger, 2010).

The purpose of this memo is to provide a review of storm water basins and wetlands where 1) designs may unintentionally promote mosquito breeding, 2) methods or designs might prevent breeding, and 3) preventative methods have been monitored for effectiveness.

## Mosquito Breeding Background

Nearly 60 species of mosquitoes are found in Kentucky and all require standing water for their development and survival from egg to emerging (flying) adult (University of Kentucky, 2011). This aquatic life history requirement has raised the concern that storm water control measures which include a standing pool of water create additional areas for mosquito breeding. Fortunately, mosquito breeding requirements are well understood and fairly restrictive, so storm water control measure designs should consider breeding information to minimize mosquito production in basins and wetlands. A brief description of the mosquito life history is provided below, with some examples of conditions found in storm water basins that provide breeding conditions.

The mosquito life cycle is generally similar for all 170 species found in North America and includes the egg, larva, pupa and adult stages (Figures 1a and 1b). However, life cycle categories of temporary pool breeders and permanent water breeders are also recognized (Hunt et al., 2005) and these categories are differentiated by the egg deposition and development adaptations. The egg stage is generally categorized by two adaptation types, 1) eggs that are deposited and incubate on soils that are continuously damp or wet (temporary or floodwater) for the duration of the egg cycle, or 2) eggs that are deposited as masses that float directly on water (permanent) for the duration of the egg cycle (Knight et al., 2003).

Temporary water species deposit their eggs in moist and quiescent areas and eggs incubate until floodwaters inundate the eggs and trigger larval hatching. The temporal success of these eggs varies by species, with some species producing eggs that can survive several years of drought with successful larval hatches (Glogoza et al., 2000). Temporary water species are commonly associated with detention facilities (dry ponds), as these facilities are intended to provide temporary seasonal and storm-related storage.

Permanent water species deposit eggs in shallow and quiescent waters such as water edges and vegetated shallows of retention basins (wet ponds) and wetlands, which as the name implies, are more permanent water features. Permanent water species generally hatch larva in 3 days (Hunt et al., 2005). The common feature for the success of hatching for both egg-depositing species is the need for wet or inundated areas to be located within quiescent portions. These quiescent areas protect eggs from the adverse wave impacts caused by wind and threats from aquatic predators such as predatory fish and other insects (Glogoza et al., 2000; Walton, 2003).

Once the egg has hatched, the larval and pupa stages follow. These stages require complete and continuous emersion in water for survival (feeding, breathing, and stage transformations). For the purpose of this memo, the larval and pupa stages are summarized together because their life-history requirements and limits are similar. The larval and pupa stages combined typically span 1 to 2 weeks (Glogoza et al., 2000). During these aquatic development stages the organisms are highly susceptible to predation. Shallow and thickly vegetated submerged areas along with floating vegetation protect the organisms from predators such as fish, predacious insects and aquatic birds (Peairs and Crenshaw, 2007). Basins and areas within basins that have poor water quality that are driven by high organic matter and nutrients (eutrophic conditions) also promote successful larval and pupa development (Walton, 2003). Mosquito larva and pupa are tolerant of poor water quality conditions; these developmental stages appear to use bacteria and algae as food sources found in quiescent, thickly vegetated areas, and poor water quality tends to deter aquatic feeding predators (Walton, 2003). Understanding the reproductive limits for this stage is important and the literature on mosquito control appears to be in general agreement with Peairs and Crenshaw (2007) who state that the key to successful mosquito management is larval habitat management.

The mature (or adult) stage is the post-pupa, flying form of the species (Glogoza et al., 2000). Adult mosquitoes typically live 2 weeks and all are strong flyers (Peairs and Crenshaw, 2007). In general, the male mosquito lives a shorter life than the female. Male mosquitoes seek fruit and flower nectar as their food source, and travel shorter flight distances from their breeding ground because their food sources tend to be associated with water (Peairs and Crenshaw, 2007). The female mosquito generally has a longer life span, feeds on nectar as well but requires a blood meal because the blood aids in egg production and development. Female mosquitoes can travel great distances to meet its blood-feeding requirements (Glogoza et al., 2000) or find suitable habitats for egg deposition (Metzger, 2004).

The adult stage is of the greatest public annoyance and of greatest concern from health-related, disease transmitting risks. Although many species cause some level of public annoyance because of their biting, host seeking and swarming behavior, relatively few mosquito species actually carry and transmit diseases of risk to humans (Knight et al., 2003). However, the risk posed by the few species that carry and transmit human affecting diseases outweighs the public tolerance to the other mosquito behaviors. Unfortunately, effective controls for adult mosquitoes are generally limited to broad-scale chemical treatments which are more difficult, costly and less effective treatments compared to those treatments focused on early life stages (Walton, 2003).

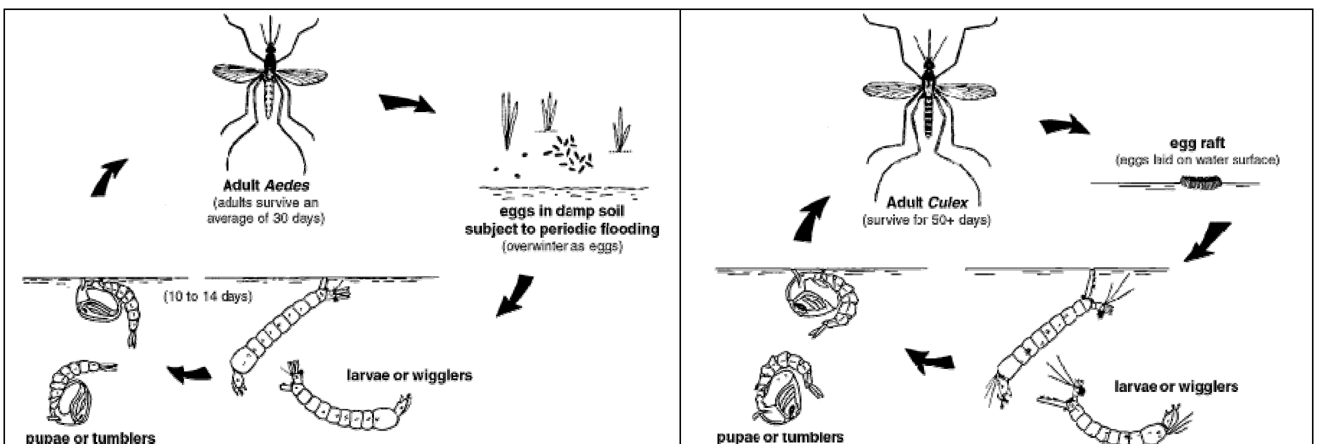


Figure 1a (left) Life cycle of temporary pool breeder, Figure 1b (right) Life cycle of permanent water breeder.

## Breeding Prevention Designs

As discussed above, storm water control measures such as retention basins and constructed wetlands may have the unintended consequence of promoting mosquito breeding (Metzger, 2004) if not properly designed. Hundreds of BMP storage designs have been tested and installed across the U.S. to meet local storm water needs (Metzger 2004). Despite the ample evidence supporting improved designs that minimize mosquito breeding, treatment facilities are still being designed and installed that function in a manner that promotes mosquito production (Walter, 2003).

Design considerations for storm water control measure designs that can influence mosquito success include the following (Metzger, 2001):

- Above or below ground installation
- Local climate
- Local fauna
- Quantity, quality and frequency of runoff
- Water table depth
- Proximity to other mosquito sources
- Mosquito host availability
- Land use
- Native and exotic vegetation
- Structural refuge (from chemical treatment)
- Maintenance commitments

These control measures require proper site selection, design, construction and maintenance to minimize mosquito production. Most literature has focused on retention basins, but the general principles discussed here will also apply to constructed wetlands with a permanent pool of water.

Retention basins and constructed wetlands are permanent aquatic systems designed to provide storm water volume storage and peak flow reductions and provide water quality improvements. The effectiveness of these control measures as "natural" water quality treatment options has resulted in a rapid expansion of the technology across the U.S. (Knight et al., 2003). These control measures are very effective for large regional storm water needs where a baseflow is available. Retention facilities are not only cost-effective solutions for water quality treatment, they also provide habitat, sites for public education and recreation, and aesthetic improvement (Walton, 2003).

Unfortunately, like their natural pond and wetland counterparts, retention basins and constructed wetlands pose management challenges because nearly all provide some level of mosquito species support (Metzger 2004). However, retention basins and constructed wetlands can be constructed and managed so that they pose a similar or even slightly lesser mosquito threat than conditions found in natural wetlands (Knight et al., 2003). The following are primary design and maintenance considerations that can limit or minimize mosquito production in retention basins and constructed wetlands:

- Maximize deeper (> 12") water areas (conversely, minimize areas < 12" deep).
- Basin embankments should be designed to be generally steep to maintain deep water conditions with limited quiescent fringes for emergent vegetation or floating vegetation (EPA, 2003)
    - 2:5 to 4:1 (horizontal:vertical) should be considered for treatment only ponds (Knight et al., 2003)
    - 5:1 to 10:1 with a narrow littoral zone shelf should be considered where habitat is also an important part of the treatment pond design.

- Wetland slopes should be designed to provide uniformly deep habitats.
- Design basins and wetlands to minimize stagnate water areas.
    - Internal berms, channels, or other structures can help circulate flows throughout basins or wetlands.
    - Large basins and wetlands should not be designed with single-point inlets; multiple inlets or inlet diffusers should be included in the design.
- Include pretreatment structures, where possible, that reduce nutrient and other pollutant inputs into facilities.
    - Improved water quality increases the likelihood that mosquito predators will successfully inhabit the constructed facility.
    - Improved water quality reduces eutrophication potential and associated nuisance algal blooms that provide food for and protect larval/pupa stages.
    - Improved water quality reduces nuisance emergent plants that protect larval/pupa stages.
- Vegetation planning is challenging in basin and wetland designs because dense, shallow, high organic conditions promote mosquito production. Vegetation design options should include the following;
    - Plant species should optimize treatment performance and mosquito control simultaneously. See Collins and Resh (1989) for compatibility guidelines.
    - Include deep water zones that are free of submergent, emergent and floating plants.
    - Shoreline vegetated areas should be maintained as narrow zones to minimize stagnant areas (EPA, 2003).

Some considerations related to operation and maintenance include the following:

- Require as-built modifications to designs and update maintenance plans accordingly.
- Establish maintenance agreements where storm water control measures will be privately owned.
- Regular maintenance of retention basins is necessary to maintain function and minimize mosquito breeding.

## Mosquito Production and Treatment Design Effectiveness

Unfortunately, many areas of research are still lacking that clarify or quantify the potential conflicts between storm water control measure design and mosquito production (Knight et al., 2003). In fact, most control measures lack any monitoring and maintenance for their treatment effectiveness let alone their potential as sources for mosquito production (Harbison et al., 2010). The following are summaries of the relatively few published studies found that have attempted to evaluate the effectiveness of detention and retention facilities on mosquito prevention.

**Study Name:** Stormwater ponds, constructed wetlands, and other best management practices potential breeding sites of West Nile virus vectors in Delaware during 2004 (Gingrich et al., 2006).
Location: Delaware

**Objective:** Evaluate and compare mosquito vector production and larval abundances at five different storm water BMP facilities.
**Designs evaluated:** Retention ponds, detention ponds, conservation enhancement and preservation program (CREP) ponds, constructed wetlands, and sand filters.
**Findings Summary:**

- 87 facilities were evaluated.
- 35 species of mosquitoes were collected.
- 5 of the species are known to transmit human disease pathogens.
- Retention basins regularly held water beyond their design period.
- Steeply banked retention basins were inversely correlated with mosquito abundance.
- The presence of mosquito predators and low shade affected mosquito abundance.
- Temporary and permanent breeder species were strongly associated with designs that associated with life history needs (that is, floodwater species dominated temporary storage facilities).
- Certain mosquito species seemed to be associated with (favored) selected aquatic vegetation types in wet detention facilities.
- Structures, such as root wads, placed in wet detention facilities as habitat features may alternatively serve as mosquito production sources.
- The facilities that produced the greatest numbers of larva throughout the season were wet facilities "choked with vegetation", with eutrophic water quality conditions.
- Shallow ponds and facilities with isolated pools were also high larva production locations.

**Study Name:** Mosquito production in stormwater treatment devices in the Lake Tahoe Basin, California (Kwan et al., 2008).
**Location:** Lake Tahoe, California
**Objective:** Document the occurrence, species composition, and seasonal abundance of mosquitoes in selected BMPs in and around the city of South Lake Tahoe.
**Designs evaluated:** Dry detention systems, sump and pump basin systems, vegetated treatment (wet) systems, and traction sand traps (winter road treatments).
**Findings Summary:**

- 47 facilities were evaluated.
- 10 species of mosquitoes were collected.
- 4 of the species are known to transmit human and animal disease pathogens.
- Standing water that exceeded design limits was observed in most dry facilities.
- Standing water in dry facilities seemed to be caused by silt and debris accumulations.
- Emergent and woody vegetation along wet pond facilities provided "ideal habitat for immature mosquitoes".
- Mosquito production in BMPs differed by season compared to natural wetland sites.
- BMPs appeared to extend the mosquito production season in sites with standing water sources.
- Temperature and rainfall determined when, where and how long mosquito breeding lasted.
- Below ground BMPs protected mosquito larva from weather extremes, potentially providing year-round production for adults.
- Frequent, storm-driven disturbances in facilities appeared to deter egg deposition in gravid females, potentially decreasing production in dry detention systems.
- Extending the dry detention design periods from 72 hours to 96 hrs will not significantly increase mosquito production in the Lake Tahoe region.
- Vector control agencies should work more closely with planners and developers to evaluate BMPs as mosquito production sites and to improve monitoring post-construction.

**Study Name:** An assessment of mosquito production and nonchemical control measures in structural stormwater best management practices in Southern California (Metzger et al., 2008).
**Location:** Los Angeles and San Diego Counties, California
**Objective:** Describe mosquito presence and relative abundance observed within individual BMPs, identify conditions conducive to mosquito production, recommend nonchemical mitigation measures and, if applied, evaluate their success.
**Designs evaluated:** 8 dry system types (n=29), 5 wet filtering system types (n=7), and 1 wet basin facility.
**Findings Summary:**

- 37 facilities were evaluated.
- 10 species were collected.
- Routine monitoring of facilities is critical for understanding facility production.
- Physical designs were primarily responsible for creating larval habitat and many designs were retrofit applications into "less-than-ideal" locations for the initial design application.
- For the period of study, Caltrans inspected all BMPs at a greater frequency than commonly conducted by "most municipal and highway operations", yet immature mosquitoes were observed even in dry systems.
- Construction errors, rises in groundwater levels and non-stormwater runoff were responsible for mosquito production in certain BMPs.
- Control measures with high mosquito production tended to be land-use related; surrounding areas with large trees, dense shrubs, and livestock had higher mosquito production than similar control measures located in relatively barren areas.
- Dry systems tended to provide a significant source of micro-habitats for larval production; for example, one rip-rapped BMP provided production conditions for 5 species of mosquitoes.
- Older structures were related to increased species use.
- Dry systems tended to attract greater species diversity than wet systems.
- Shallow, wet systems contained thick vegetation, promoting mosquito production.
- Mosquitofish were effective in the open, deeper portions of the wet basin but vegetation prevented fish access in the shallow areas, providing conditions for mosquito production.
- Multiagency collaboration is needed because agencies may have conflicting objectives (water quality verses mosquito management).
- Multiagency collaboration can result in improvements in control measure design that minimizes mosquito production.

**Study Name:** A preliminary survey for mosquito breeding in stormwater retention ponds in three Maryland counties (Dorothy and Staker, 1990).
**Location:** Prince Georges, Motgomery and Howard Counties, Maryland
**Objective:** Provide observation data on the number and types of ponds which support mosquito populations for three high-growth counties in Maryland.
**Designs evaluated:** Dry pond designs (n=83) included samples from ruts, depressions, ditches, or low areas below outflow pipes. Wet pond designs (n=56) included permanently wet stormwater ponds.
**Findings Summary:**

- 300 stormwater ponds were identified during the office review, although the total number is likely an underestimate given outdated master planning maps for the counties.
- 139 ponds were visited and monitored.
- Dry ponds
    - 70% retained water longer than design.

  - 46% contained mosquito larvae at some point in the season.
  - 18% maintained mosquito populations throughout the season.
  - 7 species of mosquitoes were collected from dry ponds.
- Wet ponds
  - 50% of the ponds contained mosquitoes.
  - 75% of the ponds with mosquitoes contained floating or emergent vegetation.
  - 10% of the ponds with mosquitoes maintained breeding populations throughout the season.
  - 6 species of mosquitoes were collected from dry ponds.
- Most ponds evaluated contained no fish and/or few predatory insects to deter mosquito production.
- Mosquito breeding in constructed ponds is likely underestimated and should be better monitored.
- Mosquito control personnel should be included in the urban planning and design process to accomplish dual objectives for stormwater facilities – water quality and mosquito controls.

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# APPENDIX K
## LIMNOTECH TECHNICAL MEMORANDUMS ON NUTRIENTS AND BACTERIA

The following Memorandums are included in this appendix:

1. Burgtorf, Stacey and Scott Bell. Nutrient Treatment Efficiency of Bioretention Cells, Constructed Wetlands, and Retention Basins. Prepared for Sanitation District No. 1. LimnoTech. 2011.

2. Habarth, Maureen, Virginia Breidenbach, and Scott Bell. Bacteria Treatment Efficiency of Constructed Wetlands and Retention Basins. Prepared for Sanitation District No. 1. LimnoTech. 2011.

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LimnoTech
Water | Environment | Scientists | Engineers

# MEMORANDUM

**DATE:** June 16, 2011

**FROM:** Stacey Burgtorf
Scott Bell

**PROJECT:** KYSDCP1A – Task 5.99

**TO:** Jim Gibson, Sanitation District No. 1 of Northern Kentucky (SD1)

**CC:** Carrie Turner (LimnoTech)
Adrienne Nemura (LimnoTech)
Project File

**SUBJECT**: Nutrient Treatment Efficiency of Bioretention Cells, Constructed Wetlands, and Retention Basins

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## Executive Summary

This memo presents a summary of findings on the treatment of nutrients from bioretention cells, constructed wetlands, and retention basins (wet ponds), based on a review of current technical literature. Total nitrogen and total phosphorus were the focus of this review. The goal of this literature review was to provide recommendations regarding nutrient treatment efficiency of these types of control measures, for use in future modeling, planning, and conceptual design work related to these watershed controls.

A total of 34 studies were reviewed, representing at least[1] 30 bioretention cells, 17 constructed wetlands, and 11 retention basins. Storm water from a variety of watershed types were treated by these systems, including agricultural, urban (residential, industrial, golf course), and laboratory settings. The most common constituents analyzed in these studies include total phosphorus and total nitrogen with nitrogen and phosphorus components also reported in some instances. This memorandum focuses on results for total nitrogen and total phosphorus only.

For all of the studies, nutrient treatment efficiency was either reported or sufficient data was provided to calculate treatment efficiency, defined as percent difference between the influent nutrient concentration and the effluent nutrient concentration. Treatment efficiencies varied widely overall with total nitrogen removal ranging from 40 to 80% for bioretention cells, -10.5 to 82.1% for constructed wetlands, and -3.7 to 57.9% for retention basins. Total phosphorus removal ranged from 4 to 99% for bioretention cells, -54 to 100% for constructed wetlands, and 15.4 to 70% for retention basins. Negative numbers indicate an increase in nutrient concentration in the effluent relative to the influent. The influence of watershed land use, influent nutrient concentrations, hydraulic retention time (HRT), and flow regime on nutrient treatment efficiency rates was also evaluated.

The reported total nitrogen and phosphorus treatment efficiencies are summarized in Tables E-1 and E-2, respectively, in terms of median values and interquartile ranges. Recommended ranges and median values for use in modeling and control measure performance calculations are also included.

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[1] Several studies were literature reviews themselves, four of which did not report the number of BMPs in their review (2 bioretention cell studies and 2 constructed wetland studies).

**Table E-1: Representative Total Nitrogen Treatment Efficiencies for Storm Water in Bioretention Cells, Constructed Wetlands, and Retention Basins**

| Statistic | Total nitrogen Treatment (%) | | |
|---|---|---|---|
| | Bioretention Cells | Constructed Wetlands | Retention Basins |
| Actual Range | 40 to 80 | -10.5 to 82.1 | -3.7 to 57.9 |
| Interquartile Range | 48.8 to 66.3 | 21 to 45.9 | 17.5 to 46.4 |
| Recommended Interquartile Range* | 50 to 65 | 20 to 45 | 20 to 50 |
| Actual Median | 57.5 | 36 | 33.5 |
| Recommended Median** | 55 | 35 | 30 |

**Table E-2: Representative Total Phosphorus Treatment Efficiencies for Storm Water in Bioretention Cells, Constructed Wetlands, and Retention Basins**

| Statistic | Total phosphorus Treatment (%) | | |
|---|---|---|---|
| | Bioretention Cells | Constructed Wetlands | Retention Basins |
| Actual Range | 4 to 99 | -54 to 100 | 15.4 to 70 |
| Interquartile Range | 54.4 to 76.1 | 32.3 to 66.5 | 26 to 53.8 |
| Recommended Interquartile Range* | 55 to 75 | 30 to 65 | 25 to 55 |
| Actual Median | 67.5 | 49.3 | 43.5 |
| Recommended Median** | 65 | 45 | 40 |

*Range presented is the interquartile range ($25^{th}$ to $75^{th}$ percentile) of the average values documented for each study, rounded to the nearest 5%.
**The recommended median represents a conservative rounding of actual medians down to the nearest 5%. It should be noted that a less conservative, but equally defensible, approach would be to round the medians up to the nearest 5%.

The lower end of the recommended ranges in Tables E-1 and E-2 represent the most conservative performance estimates based on the studies reviewed. They assume a relatively underperforming or undersized control measure. The upper end of the ranges represents a control measure that is optimally sized, with excellent design, very good maintenance, and optimum operating conditions. The recommended median value represents well-designed control measures with good maintenance and average operating conditions.

Findings of the literature review are summarized below for each control measure type:

Bioretention Cells:

- Percent treatment of total nitrogen is generally greater than 50% with a median treatment of 55% and a conservative upper treatment rate of 65%.
- Percent treatment of total phosphorus is generally greater than 55% with a median treatment of 65% and a conservative upper treatment rate of 75%.

- Bioretention cell studies took place in urban and laboratory settings. Nutrient removal rates were very similar in both settings. Bioretention cells in urban settings removed 40 to 80% of total nitrogen and 35 to 85% of total phosphorus. Bioretention cells in laboratory did not report total nitrogen removal rates. The total phosphorus removal ranged from 4 to 99%.
- The magnitude of influent concentrations did not affect nutrient treatment efficiencies. For the three control measure types as a whole, nutrient removal and influent concentration have a weak positive correlation for total nitrogen ($R^2$ = 0.006, p = 0.90, and n =5) and total phosphorus ($R^2$=0.007, p = 0.73, and n=19). The p-value was calculated for a 95% confidence interval[2].
- Bioretention cells typically have a very low HRT. The two studies reporting HRTs for bioretention cells reported a total phosphorus removal of 51.5 and 77.5% for HRTs of 7 and 7.5 hours, respectively. Removal of total nitrogen was not reported in these studies.
- Total nitrogen removal efficiencies ranged from 70 to 80% in wet weather and 40 to 80% in mixed conditions. Total phosphorus removal ranged from 35 to 50% in wet weather and 72% mixed. Synthetic flow regimes generated in laboratory settings resulted in 4 to 85% total phosphorus removal.

Constructed Wetlands:

- Percent treatment of total nitrogen is generally greater than 20% with a median treatment of 35% and a conservative upper treatment rate of 45%.
- Percent treatment of total phosphorus is generally greater than 30% with a median treatment of 45% and a conservative upper treatment rate of 65%.
- Constructed wetland studies took place primarily in agricultural or urban watersheds. Overall, 27 to 52% of total nitrogen and 20 to 68% of total phosphorus was removed in agricultural watersheds. In urban watersheds, -10.5 to 80% of total nitrogen and 12.5 to 100% of total phosphorus were removed. Overall, the range of treatment is wider for urban settings than agricultural.
- The magnitude of influent concentrations did not affect nutrient treatment efficiencies. For the three control measure types as a whole, nutrient removal and influent concentration have a weak positive correlation for total nitrogen ($R^2$ = 0.006, p = 0.90, and n =5) and total phosphorus ($R^2$=0.007, p = 0.73, and n=19). The p-value was calculated for a 95% confidence interval.
- Constructed wetlands typically have a long retention time on the order of days. Nutrient removal and HRT have a weak positive correlation for total nitrogen ($R^2$ = 0.013, p = 0.81, and n=7) and total phosphorus ($R^2$ = 0.45, p = 0.012, and n=13). The low $R^2$ value and high p-value indicate a low statistical significance for this correlation.
- Total nitrogen removal efficiencies ranged from -10.5 to 80% in dry weather, 20 to 82.1% in wet weather, and 20 to 82.1% in mixed conditions. Total phosphorus removal efficiencies ranged from 12.5 to 100% in dry weather, -18.2 to 90% in wet weather, and -54 to 83.5% in mixed conditions.

[2] A high p-value indicates a lack of statistical significance, and P-values less than 0.05 indicate a high degree of statistical significance.

Retention Basins:

- Percent treatment of total nitrogen is generally greater than 20% with a median treatment of 30% and a conservative upper treatment rate of 50%.
- Percent treatment of total phosphorus is generally greater than 25% with a median treatment of 40% and a conservative upper treatment rate of 55%.
- Retention basin studies took place in urban settings. Total nitrogen treatment ranged from -3.7 to 57.9%, and total phosphorus treatment ranged from 15.4 to 70%.
- The magnitude of influent concentrations did not affect nutrient treatment efficiencies. For the three control measure types as a whole, nutrient removal and influent concentration have a weak positive correlation for total nitrogen ($R^2$ = 0.006, p = 0.90, and n =5) and total phosphorus ($R^2$=0.007, p = 0.73, and n=19). The p-value was calculated for a 95% confidence interval. A high p-value indicates a lack of statistical significance, and P-values less than 0.05 indicate a high degree of statistical significance.
- No reviews of retention basins reported hydraulic retention times.
- Total nitrogen removal efficiencies were reported as 57.9% for dry weather, 42.5% for wet weather, and -3.7 to 24.5 for mixed conditions. Total phosphorus removal efficiencies were reported as 15.4% for dry weather, 70% for wet weather, and 20 to 54.4 for mixed conditions.

The relatively higher treatment efficiency of bioretention cells compared to retention basins or constructed wetlands suggest that the up-scaling bioretention techniques to retrofit detention basins (dry ponds) might be an effective control strategy for nutrients.

## Overview

This memorandum presents the findings of a literature review of nutrient treatment rates for bioretention cells, constructed wetlands, and retention basins, three types of control measures commonly used for storm water treatment. The focus of this review was to evaluate the effectiveness of these control measures in removing total nitrogen and total phosphorus from storm water as reported in the literature. The results of this review could be used to guide the selection of nutrient treatment efficiencies for use in modeling or for use in control measure performance calculations.

The findings of this review are presented in the following sections:

Description of control measures Examined

Overview of Studies Reviewed

Key Findings

Summary

References

Attachment A: Detailed Study Results

## Description of Control Measures Examined

Nutrient treatment efficiency in bioretention cells, constructed wetlands, and retention basins was examined in this literature review. Basic descriptions of these treatment methods are provided below.

### *Bioretention Cells*

Bioretention cells are small landscape depressions into which storm water runoff is diverted and stored. Once in the cell, the trees, shrubs, and other vegetation help to remove the water through uptake, and the rest infiltrates into the soil. The underlying soil may consist of the original soil, but typically is a non-native, well-infiltrating soil that is installed during construction. The soil media typically consists of a surface layer of hardwood mulch, followed by a vegetated layer supported by a porous soil type such as sand (Hsieh and Davis, 2005). Pollution removal occurs at the surface and in the deeper soil media layers, making the bioretention cell essentially a vegetated sand filter (Hunt et al., 2006). Bioretention cells may also include a perforated underdrain that collects and removes infiltrated water (Weiss et al., 2007). The use of bioretention cells has increased in recent years due to their aesthetic appeal and the small area they occupy, which allows for the use of this control measure in urban areas with limited land space.

### *Constructed Wetlands*

Constructed wetlands are wide, shallow storage areas containing a significant amount of vegetation. They typically hold a permanent pool of water so that wetland vegetation can be maintained. Constructed wetlands can take several days or more to release storm events, allowing for long contact times. Constructed wetlands typically require large areas to allow for adequate storage volumes and long flow paths (Weiss et al., 2007).

There are three primary types of treatment wetlands defined by the flow of water through the wetland: natural, surface flow, and subsurface flow. Variations on these primary types of treatment wetlands include horizontal flow and vertical flow. By choosing the type of treatment wetland or hybrid wetland system appropriate for the application, it is possible to achieve a high level of treatment for a variety of pollutants including bacteria, nutrients, metals, and solids. Treatment wetlands can also provide additional environmental and community benefits, including habitat creation, water level control, aesthetic enhancement, and public education opportunities (LimnoTech, 2011).

### *Retention Basins*

Retention basins detain storm water runoff for a period of hours or days, while releasing it to receiving streams and lakes. Although they are designed primarily for water-quantity increases, they are now being used to reduce non-point pollution. Retention basins maintain a permanent pool of water, extending the residence time relative to detention basins. They are typically deeper than constructed wetlands, occupy less surface area, and have shorter residence times (Comings et al., 2000).

Retention basins are generally designed to reduce suspended sediments. However, these basins can effectively reduce bacteria densities, as well as concentrations of other pollutants such as nutrients and heavy metals. It has been reported that the level of treatment in a retention basin is dependent primarily on basin geometry (Mallin et al., 2002).

## Overview of Studies Reviewed

A literature search was conducted in order to summarize the performance of bioretention cells, constructed wetlands, and retention basins for nutrient treatment. Numerous studies have reviewed the potential for these systems to remove nutrients from storm water associated with agricultural, urban (residential, industrial, golf course), and laboratory settings. The majority of studies focused on agricultural and urban watersheds.

A total of 34 studies were reviewed, representing at least 30 bioretention cells, 17 constructed wetlands, and 11 retention basins. Most of these studies were conducted in either North America or Europe. All 23 of the North American studies reviewed were conducted in the United States. In Europe, six studies were performed in five different countries: Denmark, Finland, Ireland, Norway, and Switzerland, with Switzerland having two studies. In Asia, two studies were conducted in China and one in both Malaysia and Taiwan. Table 1 summarizes the geographic locations of the reviewed studies by continent.

**Table 1**: **Count of Studies Reviewed by Continent of Study Location**

| Continent | # Studies |
|---|---|
| Asia | 4 |
| Australia | 1 |
| Europe | 6 |
| North America | 23 |
| Total | 34 |

Total phosphorus was the most common nutrient analyzed in the reviewed studies, followed by total nitrogen, nitrate, ammonium, ammonia, phosphate, and others. Table 2 summarizes the nitrogen and phosphorus constituents analyzed and the number of studies reporting results for each type. Some of the studies reviewed reported treatment efficiencies for other nutrients as well as bacteria; those results and as well as components of phosphorus and nitrogen are not included in this review. The studies summarizing results for total phosphorus and total nitrogen are the focus of this review.

**Table 2**: **Studies Reviewed by Type of Nitrogen and Phosphorus Constituents Analyzed**

| Constituent | # Studies |
|---|---|
| Total Phosphorus | 28 |
| Total Nitrogen | 18 |
| Nitrate | 12 |
| Ammonium | 8 |
| Ammonia | 5 |
| Phosphate | 4 |
| Total Kjeldahl Nitrogen | 4 |
| Dissolved Reactive Phosphorus | 3 |
| Nitrite | 2 |
| Orthophosphorus | 2 |
| Reactive Phosphorus | 2 |
| Soluble Reactive Phosphorus | 2 |
| Molybdate Reactive Phosphorus | 1 |
| Nitrate+Nitrite | 1 |
| Nitrogen Dioxide | 1 |
| Organic Nitrogen | 1 |
| Orthophosphate-Phosphorus | 1 |
| Particulate Nitrogen | 1 |
| Particulate Organic Nitrogen | 1 |
| Particulate Organic Phosphorus | 1 |

Watershed land uses were reported for each study, with some studies analyzing control measures in several watersheds with differing land uses. Table 3 summarizes the types of watershed land uses that each control measure type was placed within. Bioretention cells were placed in urban watersheds or simulated in laboratory settings. Constructed wetlands were placed in urban, agricultural, mixed, and laboratory settings. Retention basins were placed only in urban settings for the studies reviewed here.

Hydraulic retention times (HRTs) were reported or calculated for 16 of the 34 studies. HRTs were reported for two bioretention cell studies and 16 constructed wetland studies, with bioretention cells having a much lower HRT than constructed wetlands, as shown in Table 4.

**Table 3: Studies Reviewed by Land Use and Control Measure Type**

| Control Measure | Land Use | # Studies |
|---|---|---|
| Bioretention Cells | Urban | 5 |
| | Lab Experiments | 3 |
| Constructed Wetlands | Urban | 16 |
| | Agricultural and Urban | 13 |
| | Agricultural | 10 |
| | Lab Experiments | 2 |
| Retention Basins | Urban | 10 |

**Table 4: Range of Hydraulic Retention Times (HRTs) Reported**

| Control Measure Type | HRT |
|---|---|
| Bioretention Cell | 6 to 7.5 hours |
| Constructed Wetland | 25.8 hrs to 27 days |

## Key Findings

This section presents key findings regarding nutrient treatment from bioretention cells, constructed wetlands, and retention basins based on the review of treatment rates reported in the literature. More detail on individual study results, including descriptions of the various conditions impacting nutrient treatment in those studies, is provided in Attachment A.

The findings are presented in the following subsections:

Range of Reported Nutrient Treatment Efficiencies

Effect of Watershed Land Use

Effect of Influent Concentration

Effect of Hydraulic Retention Time

Effect of Flow Regime

Range of Recommended Nutrient Treatment Efficiencies

### *Range of Reported Nutrient Treatment Efficiencies*

A wide range of nutrient treatment efficiencies was reported for all control measure treatment systems reviewed.

- Few studies were available on nutrient treatment by bioretention cells. Total nitrogen treatment in bioretention cells ranged from 40 to 80%. Total phosphorus treatment ranged from 4 to 99%.
- A very wide range of nutrient treatment efficiencies was reported for constructed wetlands. Total nitrogen removal was reported as high as 82.1% and as low as -10.5%. Total phosphorus removal ranged from -54 to 100%.
- Studies reviewing retention basin nutrient removal efficiencies also indicate a wide range. Removal rates between -3.7 and 57.9% were reported for total nitrogen and 15.4 to 70% for total phosphorus.

These ranges indicate that nutrient concentrations can either increase or decrease in control measures depending on system design, operating conditions, and environmental factors.

### *Effect of Watershed Land Use*

Reported nutrient efficiencies for various watershed land uses for bioretention cells, constructed wetlands, and retention basins are shown in Table 5. Studies reported nutrient removal for agricultural, urban, and mixed conditions. Synthetic conditions, such as those created in a laboratory setting, were analyzed as well. In general, the range of treatment efficiency for urban watershed is wider than for agricultural watersheds. Also, the range in total phosphorus removal efficiencies is greater than that of total nitrogen for all control measure types. Laboratory settings were only associated with bioretention cells, which have a wide range of total phosphorus removal efficiencies.

**Table 5: Reported Nutrient Treatment Efficiencies by Control Measure Type and Watershed Land Use**

| | | **Nutrient Treatment (%)** | |
|---|---|---|---|
| **Control Measure** | **Land Use** | **TN** | **TP** |
| Bioretention Cells | Agricultural | N.A. | N.A. |
| | Urban | 40 to 80 | 35 to 85 |
| | Agricultural and Urban | N.A. | N.A. |
| | Lab Experiments | N.A. | 4 to 99 |
| Constructed Wetlands | Agricultural | 27 to 52 | 20 to 68 |
| | Urban | 0 to 80 | 12.5 to 100 |
| | Agricultural and Urban | 16 to 82.1 | 0 to 92 |
| | Lab Experiments | 40 to 55 | 40 to 60 |
| Retention Basins | Agricultural | N.A. | N.A. |
| | Urban | 0 to 57.9 | 15.4 to 70 |
| | Agricultural and Urban | N.A. | N.A. |
| | Lab Experiments | N.A. | N.A. |

### *Effect of Influent Concentration*

The studies reviewed do not support a solid conclusion regarding the relationship between nutrient treatment efficiency and influent concentrations. A higher influent concentration may result in the same percent treatment as a lower influent concentration. Where paired influent concentration rates and removal efficiencies were reported for total nitrogen and total phosphorus, the influence of influent concentrations on reported treatment rates was investigated. For total nitrogen, there was one reported pairing for bioretention cells, two for constructed wetlands, and two for retention basins. For total phosphorus, there were 5 reported pairings for bioretention cells, 12 for constructed wetlands, and two for retention basins. Overall, 14 studies reported nutrient treatment efficiency and influent concentration pairings.

For the control measures as a whole, nutrient removal rates and influent concentrations were very weakly correlated, $R^2 = 0.006$, $p = 0.90$, and $n = 5$ for total Nitrogen and $R^2 = 0.007$, $p = 0.73$, and $n = 19$ for total Phosphorus. The p-value was calculated for a 95% confidence interval. Figure 1 shows that the data is scattered, showing neither an increasing or decreasing pattern.

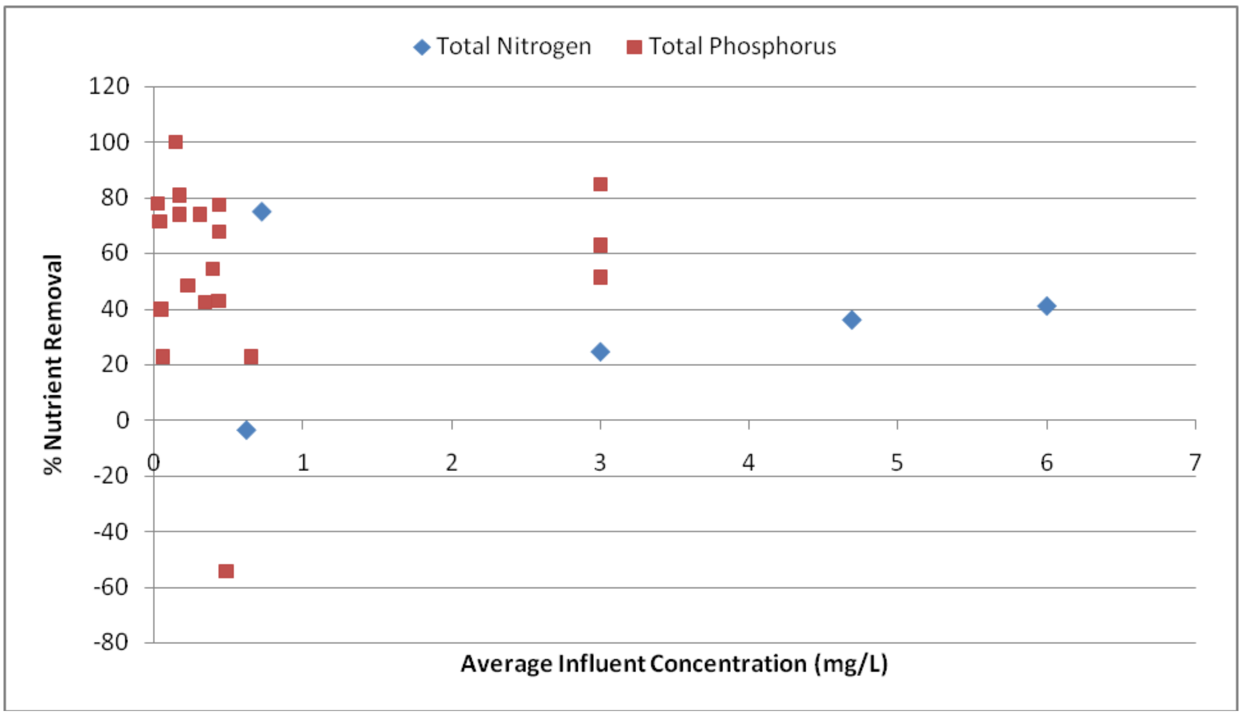


**Figure 1: Influence of Influent Concentration on Nutrient Treatment in Constructed Wetlands**

### *Effect of Hydraulic Retention Time*

Several studies reported differing treatment efficiencies for similar HRTs. The relationship between HRT and nutrient treatment rates was investigated for the studies reporting both parameters (two studies for bioretention cells and 14 studies for constructed wetlands). Bioretention cells typically have very short HRTs, on the order of a few hours. The two studies reporting HRTs for bioretention cells had HRTs of 6 and 7.5 hours for an average total phosphorus removal of 51.5 to 77.5%, respectively. Constructed wetlands typically have long retention times, on the order of days. A regression analysis was conducted to determine the strength of the relationship between nutrient removal and HRT for constructed wetlands, because sufficient data were available for that control measure type. The correlation coefficient between HRT and percent treatment of total nitrogen is 0.11, indicating a weak positive correlation between HRT and nutrient removal rates. Increased hydraulic retention times generally lead to increased total nitrogen removal. Nutrient removal and HRT have a weak positive correlation for total nitrogen ($R^2$ = 0.013, p = 0.81, and n=7) and total phosphorus ($R^2$ = 0.45, p = 0.12, and n=13). The low $R^2$ value and high p-value indicate a low statistical significance for this correlation. No studies reviewing retention basins reported HRTs. In general, the studies reviewed do not support a solid relationship between HRT and nutrient removal efficiency.

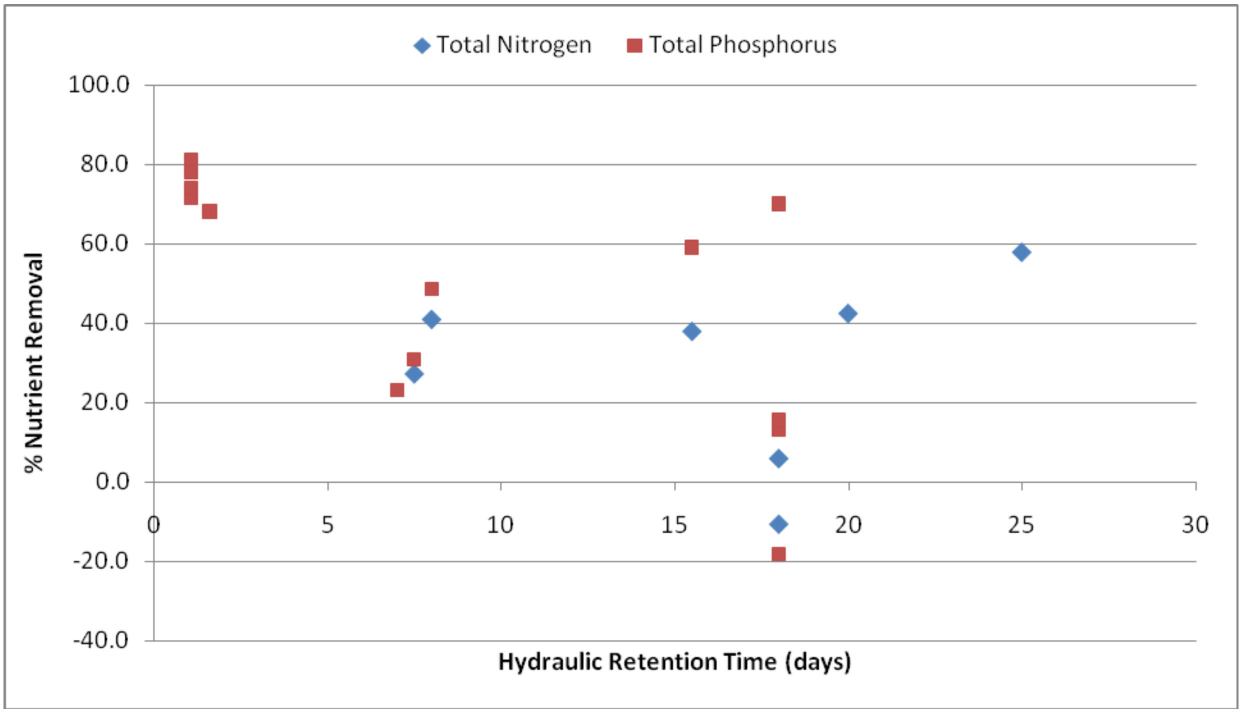


**Figure 2: Influence of Hydraulic Retention Time on Nutrient Treatment in Constructed Wetlands**

### *Effect of Flow Regime*

Reported nutrient efficiencies for various flow regimes for bioretention cells, constructed wetlands, and retention basins are shown in Table 6. Studies reported nutrient removal for dry and wet weather conditions as well as overall values for the entire study, which generally includes a range of hydrologic conditions. Synthetic conditions, such as those created in a laboratory setting, were analyzed as well. In general, the range in treatment efficiency for total nitrogen is quite similar for each flow regime, with negative removal efficiencies (accrual) for dry weather conditions. For total phosphorus, negative removal efficiencies are associated with wet weather conditions. Bioretention cells in laboratory conditions exhibited a wide range of total phosphorus removal efficiencies, 4 to 99%. No laboratory conditions were associated with constructed wetlands or retention basins.

**Table 6: Reported Nutrient Treatment Efficiencies by Control Measure Type and Flow Regime**

| Control Measure | Flow Regime | Nutrient Treatment (%) TN | TP |
|---|---|---|---|
| Bioretention Cells | Dry Weather | N.A. | N.A. |
| | Wet Weather | 70 to 80 | 35 to 50 |
| | Overall | 40 to 80 | 72 |
| | Lab Experiments | N.A. | 4 to 85 |
| Constructed Wetlands | Dry Weather | -10.5 to 80 | 12.5 to 100 |
| | Wet Weather | 20 to 82.1 | -18.2 to 90 |
| | Overall | 20 to 82.1 | -54 to 83.5 |
| Retention Basins | Dry Weather | 57.9 | 15.4 |
| | Wet Weather | 42.5 | 70 |
| | Overall | -3.7 to 24.5 | 20 to 54.4 |

### *Range of Recommended Nutrient Treatment Efficiencies*

The interquartile range has been used by others (Schueler, et al., 2007) to describe the range of treatment efficiencies considered to be appropriate for use when evaluating variations in pollutant treatment expected from control measures. Properly designed and maintained control measures might be given a higher value in this range, while underperforming or undersized control measures might be given a lower value in this range.

Based on the analyses and review of study results presented in this and preceding sections, a range of representative nutrient treatment efficiencies was selected for each control measure type for performance calculations and modeling purposes based on the interquartile results, as indicated in Tables 7 and 8.

**Table 7: Representative Total nitrogen Treatment Efficiencies for Storm Water in Bioretention Cells, Constructed Wetlands, and Retention Basins**

| Statistic | Total nitrogen Treatment (%) | | |
|---|---|---|---|
| | Bioretention Cells | Constructed Wetlands | Retention Basins |
| Actual Range | 40 to 80 | -10.5 to 82.1 | -3.7 to 57.9 |
| Interquartile Range | 48.8 to 66.3 | 21 to 45.9 | 17.5 to 46.4 |
| Recommended Interquartile Range* | 50 to 65 | 20 to 45 | 20 to 50 |
| Actual Median | 57.5 | 36 | 33.5 |
| Recommended Median** | 55 | 35 | 30 |

**Table 8: Representative Total phosphorus Treatment Efficiencies for Storm Water in Bioretention Cells, Constructed Wetlands, and Retention Basins**

| Statistic | Total phosphorus Treatment (%) | | |
|---|---|---|---|
| | Bioretention Cells | Constructed Wetlands | Retention Basins |
| Actual Range | 4 to 99 | -54 to 100 | 15.4 to 70 |
| Interquartile Range | 54.4 to 76.1 | 32.3 to 66.5 | 26 to 53.8 |
| Recommended Interquartile Range* | 55 to 75 | 30 to 65 | 25 to 55 |
| Actual Median | 67.5 | 49.3 | 43.5 |
| Recommended Median** | 65 | 45 | 40 |

*Range presented is the interquartile range (25$^{th}$ to 75$^{th}$ percentile) of the average values documented for each study, rounded to the nearest 5%.
**The recommended median represents a conservative rounding of actual medians down to the nearest 5%. It should be noted that a less conservative, but equally defensible, approach would be to round the medians up to the nearest 5%.

The lower end of the recommended ranges in Tables 7 and 8 represent the most conservative performance estimate based on the studies reviewed. It assumes a relatively underperforming or undersized control measure. The upper end of the range represents a control measure that is optimally sized, with excellent design, very good maintenance, and optimum operating conditions. The recommended median value represents well-designed control measures with good maintenance and average operating conditions.

## Summary

Nutrient treatment in bioretention cells, constructed wetlands, and retention basins was investigated through a review of available literature. A total of 34 studies were reviewed, representing at least[3] 30 bioretention cells, 17 constructed wetlands, and 11 retention basins. Treatment efficiencies, defined as the percent difference between the influent and effluent nutrient concentration, were reported or calculated for all of the studies.  Treatment efficiencies varied widely overall with total nitrogen removal ranging from 40 to 80% for bioretention cells, -10.5 to 82.1% for constructed wetlands, and -3.7 to 57.9% for retention basins. Total phosphorus removal ranged from 4 to 99% for bioretention cells, -54 to 100% for constructed wetlands, and 15.4 to 70% for retention basins. Negative numbers indicate an increase in nutrients of the effluent relative to the influent.

Where data were available, the influence of watershed land use, influent concentration, hydraulic retention time, and flow regime on nutrient treatment was evaluated. Based on these analyses, a range of representative total nitrogen and total phosphorus efficiencies was selected for each control measure type for design and modeling purposes. The following statements can be made regarding nutrient treatment in bioretention cells, constructed wetlands, and retention basins:

[3] Several studies were literature reviews themselves, four of which did not report the number of BMPs in their review (2 bioretention cell studies and 2 constructed wetland studies).

Bioretention Cells:

- Percent treatment of total nitrogen is generally greater than 50% with a median treatment of 55% and a conservative upper treatment rate of 65%.
- Percent treatment of total phosphorus is generally greater than 55% with a median treatment of 65% and a conservative upper treatment rate of 75%.
- Bioretention cell studies took place in urban and laboratory settings. Nutrient removal rates were very similar in both settings. Bioretention cells in urban settings removed 40 to 80% of total nitrogen and 35 to 85% of total phosphorus. Bioretention cells in laboratory did not report total nitrogen removal rates. The total phosphorus removal ranged from 4 to 99%.
- The magnitude of influent concentrations did not affect nutrient treatment efficiencies. For the three control measure types as a whole, nutrient removal and influent concentration have a weak positive correlation for total nitrogen ($R^2$ = 0.006, p = 0.90, and n =5) and total phosphorus ($R^2$=0.007, p = 0.73, and n=19). The p-value was calculated for a 95% confidence interval[4].
- Bioretention cells typically have a very low HRT. The two studies reporting HRTs for bioretention cells reported a total phosphorus removal of 51.5 and 77.5% for HRTs of 7 and 7.5 hours, respectively. Removal of total nitrogen was not reported in these studies.
- Total nitrogen removal efficiencies ranged from 70 to 80% in wet weather and 40 to 80% in mixed conditions. Total phosphorus removal ranged from 35 to 50% in wet weather and 72% mixed. Synthetic flow regimes generated in laboratory settings resulted in 4 to 85% total phosphorus removal.

Constructed Wetlands:

- Percent treatment of total nitrogen is generally greater than 20% with a median treatment of 35% and a conservative upper treatment rate of 45%.
- Percent treatment of total phosphorus is generally greater than 30% with a median treatment of 45% and a conservative upper treatment rate of 65%.
- Constructed wetland studies took place primarily in agricultural or urban watersheds. Overall, 27 to 52% of total nitrogen and 20 to 68% of total phosphorus was removed in agricultural watersheds. In urban watersheds, -10.5 to 80% of total nitrogen and 12.5 to 100% of total phosphorus were removed. Overall, the range of treatment is wider for urban settings than agricultural.
- The magnitude of influent concentrations did not affect nutrient treatment efficiencies. For the three control measure types as a whole, nutrient removal and influent concentration have a weak positive correlation for total nitrogen ($R^2$ = 0.006, p = 0.90, and n =5) and total phosphorus ($R^2$=0.007, p = 0.73, and n=19). The p-value was calculated for a 95% confidence interval.
- Constructed wetlands typically have a long retention time on the order of days. Nutrient removal and HRT have a weak positive correlation for total nitrogen ($R^2$ = 0.013, p = 0.81, and n=7) and total phosphorus ($R^2$ = 0.45, p = 0.012, and n=13). The low $R^2$ value and high p-value indicate a low statistical significance for this correlation.

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[4] A high p-value indicates a lack of statistical significance, and P-values less than 0.05 indicate a high degree of statistical significance.

- Total nitrogen removal efficiencies ranged from -10.5 to 80% in dry weather, 20 to 82.1% in wet weather, and 20 to 82.1% in mixed conditions. Total phosphorus removal efficiencies ranged from 12.5 to 100% in dry weather, -18.2 to 90% in wet weather, and -54 to 83.5% in mixed conditions.

Retention Basins:

- Percent treatment of total nitrogen is generally greater than 20% with a median treatment of 30% and a conservative upper treatment rate of 50%.
- Percent treatment of total phosphorus is generally greater than 25% with a median treatment of 40% and a conservative upper treatment rate of 55%.
- Retention basin studies took place in urban settings. Total nitrogen treatment ranged from -3.7 to 57.9%, and total phosphorus treatment ranged from 15.4 to 70%.
- The magnitude of influent concentrations did not affect nutrient treatment efficiencies. For the three control measure types as a whole, nutrient removal and influent concentration have a weak positive correlation for total nitrogen ($R^2$ = 0.006, p = 0.90, and n =5) and total phosphorus ($R^2$=0.007, p = 0.73, and n=19). The p-value was calculated for a 95% confidence interval. A high p-value indicates a lack of statistical significance, and P-values less than 0.05 indicate a high degree of statistical significance.
- No reviews of retention basins reported hydraulic retention times.
- Total nitrogen removal efficiencies were reported as 57.9% for dry weather, 42.5% for wet weather, and -3.7 to 24.5 for mixed conditions. Total phosphorus removal efficiencies were reported as 15.4% for dry weather, 70% for wet weather, and 20 to 54.4 for mixed conditions.

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# Attachment A: Detailed Study Results

## Summary of Individual Nutrient Treatment Study Results

Bioretention cell, constructed wetland, and retention basin performance in removing nutrients from storm water is dependent on many factors, as evidenced by the studies reviewed. The focus of the individual studies varied greatly. For example, some studies examined the performance of control measures placed in watersheds of different land uses, or control measures treating low or high flow events. Some studies focused on the system configuration, such as hydraulic retention time, geometry, loading rate, and influent concentration, while others focused on vegetation type or substrate material. Several focused on seasonal differences in performance over several years of monitoring.

Treatment of total nitrogen and total phosphorus was typically reported as an average percent treatment over the monitoring period, for all hydraulic or temporal conditions. It was calculated based on the difference in nutrient concentration measured at the inlet and outlet of the system. The following is a discussion of study results for bioretention cells, constructed wetlands, and retention basins.

### *Bioretention Cells*

Six of the studies reviewed focused on the use of bioretention cells for the treatment of total nitrogen and total phosphorus from storm water runoff. Reported treatment efficiencies varied widely, suggesting that nutrient treatment in these systems is sensitive to design, operation, and environmental conditions. Additional research is likely needed to further characterize the capability of bioretention cells to remove nutrients.

Two studies reported total nitrogen removal rates for bioretention cells in urban watersheds, ranging from 40 to 80%. Hunt et al. (2006) reported an annual average removal of 40%, while Smith and Hunt (2007) reported a higher removal rate of 70-80% during their monitoring of wet weather events over a six month period.

Reported removal efficiencies for total phosphorus ranged from 4 to 99%. This wide range in total phosphorus treatment efficiencies was documented by Hsieh and Davis (2005). Eighteen bioretention columns and six existing bioretention facilities were evaluated employing synthetic runoff. The authors indicated that the wide range in phosphorus removal was due to preferential flow patterns. Media depth and texture was correlated with total phosphorus removal, but no significant relationship was found. Laboratory experiments from other sources indicate a much narrower range of total phosphorus removal, 63-85%, with high hydraulic conductivity media as the top layer having increased treatment relative to lower conductivity media. Wet weather monitoring in of a bioretention cell in an urban watershed by Smith and Hunt (2007) resulted in a removal rate of 35-50%. This narrow range may be due to the fact that there was only one flow regime during sampling. Weiss et al. (2007) reported an overall removal efficiency of 72% for the studies he reviewed over a 20 year period.

Overall, the studies reviewed demonstrate that there is a high degree of variability in terms of nutrient removal performance among bioretention cells. The most influential factors in treatment capacity appear to be land use characteristics, conductivity of top layer media, and flow regimes (wet weather versus annual average).

The studies reviewed pertaining to the effectiveness of bioretention cells for nutrient treatment are summarized in Table A-1.

**Table A-1: Summary of Studies Reviewed on Bioretention Cell Performance for Nutrient Treatment**

| Reference | Project Timeframe | Watershed Land Use | HRT (days) | Conditions | Influent Concentration (mg/L) | | Nutrient Treatment (%) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | TN | TP | TN | TP |
| Davis et al., 2006 | < 1 year | Urban | 0.125-0.5 | Lab and field experiments | | 0.40-0.48 | | 70-85 |
| Hsieh and Davis, 2005 | 6 hrs | Laboratory | 0.25 | Lab and field experiments | | 3 | | 4-99 |
| Hsieh et al., 2007 | 29 days | Laboratory | | Lab experiment, high conductivity media over low | | 3 | | 85 |
| | | | | Lab experiment, low conductivity media over high | | 3 | | 63 |
| Hunt et al., 2006 | 1 year | Urban | | Overall | | | 40 | |
| Smith and Hunt, 2007 | 6 months | Urban | | Wet Weather | 0.25-1.2 | 0.18-0.51 | 70-80 | 35-50 |
| Weiss et al., 2007 | 20-year | Urban | | Overall | | | | 72 |

## *Constructed Wetlands*

Twenty of the studies reviewed focused on the use of constructed wetlands for the treatment of total nitrogen and total phosphorus from storm water runoff. Reported treatment efficiencies varied widely, from negative percent removals (nutrient accrual) to complete removal. This suggests that nutrient treatment in these systems is sensitive to design, operation, and environmental conditions.

Fourteen studies reported total nitrogen removal rates ranging from -10.5 to 82.1%. Greenway (2010) recorded a -10.5% removal of nitrogen during dry weather and 6.3% in wet weather. Low or even negative removal, an increase in nutrient concentration in the effluent relative to the influent, was attributed to several factors. For dry weather, resuspension of organic particles can occur from the substrate to the water column, increasing the effluent concentration of nutrients. For wet weather, storm water flushes out stored water and associated organic matter and nutrients. Greenways stated that this flush-out effect tends to be present in non-vegetated basins as well, but to a lesser extent. Studies reporting overall nutrient removal efficiencies for the course of the study, rather than for specific events or seasons, report at least a 20% removal of total nitrogen with an upper bound of 82.1%.

Nineteen studies reported total phosphorus removal efficiencies in the range of -54 to 100%. The range of total phosphorus removal is even wider than that of total nitrogen. Bass (2000) reported an overall phosphorus removal efficiency of -54% for his two year study of a constructed wetland in an agricultural and urban watershed. Bass attributed the increase in phosphorus concentrations to inputs from a small storm drain, associated with a high proportion of impervious surface area that drains to the wetland. Waste from wildlife may also contribute, as this wetland was a popular lounging area for ducks. Nitrate-nitrogen levels did not increase, indicating that waterfowl waste may be the primary cause of the increase. Greenway (2010) also recorded a negative total phosphorus removal rate, -18.2%, during wet weather. Again, this was attributed to storm water flows flushing out nutrients from the existing water in the wetland. Kohler et al. (2004) reported much higher removal efficiencies of total phosphorus: 74% during storm conditions and 100% for baseflow conditions. This wetland was located on an urban golf course fed by direct runoff and a tile drainage system. Higher removal rates were attributed to low influent phosphorus concentrations. Of the 83 chemicals monitored by the study by Kohler, only 17 had a measureable concentration. The other 16 studies also report a wide range of phosphorus removal, 12.5 to 90%.

Overall, these studies demonstrate a very high degree of variability in terms of nutrient treatment performance among constructed wetlands. While some wetlands were able to achieve very high nutrient treatment, others increased nutrient concentrations during the monitoring period. The most influential factors in treatment capacity seem to be land use characteristics, flow regime (wet, dry, or average), and presence of direct sources of nutrients such as pipe inflow or waterfowl.

The studies reviewed pertaining to the effectiveness of constructed wetlands for nutrient treatment are summarized in Table A-2.

## Table A-2: Summary of Studies Reviewed on Constructed Wetland Performance for Nutrient Treatment

| Reference | Project Timeframe | Watershed Land Use | HRT (days) | Conditions | Influent Concentration (mg/L) | | Nutrient Treatment (%) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | TN | TP | TN | TP |
| Bass, 2000 | 2 years | Agricultural and Urban | | Overall | | 0.29-0.69 | 20 | -54 |
| Coveney et al., 2002 | 6 months | Agricultural | 4-27, usually < 8 | Overall | 3-9 | 0.08-0.38 | 30-52 | 30-67 |
| Fink and Mitsch, 2004 | 2 years | Agricultural | | Overall | | | | 59 |
| | | | | Wet Weather | | | | 28 |
| Greenway, 2010 | 3 years | Urban | 8 in wetland 1 and 16-20 in wetlands 1 and 2 | Wet Weather | | | 6.3 | -18.2 |
| | | | | Dry Weather | | | -10.5 | 12.5 |
| Gu, 2008 | 2 years | Agricultural | | Overall | | 0.023-0.075 | | 20-60 |
| Henry et al., 2003 | 3 years | Open Lot | | Overall | | | | 83.5 |
| Johnson, 2007 | 14 months | Urban | | Wet Weather | 0.39-8.98 | 0.02-0.85 | 36 | 43 |
| Jordan et al., 2003 | 1 year | Agricultural | 19 | Overall | | | 38 | 59 |
| Kohler et al., 2004 | 4 years | Urban | | Wet Weather | | 0.31 | | 74 |
| | | | | Dry Weather | | 0.15 | | 100 |
| Li et al., 2009 | 2 years | Urban | 7.5 | Overall | | | 27.3 | 30.8 |
| Liikanen et al., 2004 | 3 years | Agricultural | 1.6 | Overall | | 0.44 | | 68 |
| Liu et al, 2008 | 1990-2007 | | | Overall | | | 44.3 | 62.1 |
| Ludwig, 2010 | <1 year | Agricultural and Urban | | Fall | | | 16 | 37 |
| | | | | Spring | | | 22 | 25 |
| Mitsch et al., 1995 | 3 years | Agricultural and Urban | 1.08 | Dry Weather | | 0.0172-0.0435 | | 64-92 |
| | | | | Wet Weather Event | | 0.02264-0.6333 | | 53-90 |
| | 1 year | Agricultural and Urban | 1.08 | Dry Weather | | 0.176 | | 81 |
| | | | | Wet Weather | | 0.176 | | 74 |
| Rea, 2004 | 1 year | Urban | | Dry Weather | | | 80 | 60 |
| Reinhardt et al., 2005 | 2 years | Agricultural | 7 | Overall | | 0.01-1.3 | 27 | 23 |
| Sim et al., 2008 | 17.5 months | Agricultural and Urban | | Overall | | | 82.1 | |
| Vymazal, 2007 | 1965-2005 | | | Overall | | | 40-55 | 40-60 |
| Weiss et al., 2007 | 20 years | Urban | | Overall | | | | 42 |
| Woodruff, 2005 | 1 year | Urban | | Dry Weather | | | 70 | 55 |

## *Retention Basins*

Five of the studies reviewed focused on the use of retention basins for the treatment of total nitrogen and total phosphorus from storm water runoff. Reported treatment efficiencies varied widely, from negative percent removals (nutrient accrual) to complete removal. This suggests that nutrient treatment in these systems is sensitive to design, operation, and environmental conditions.

Five studies reported total nitrogen removal rates ranging from -3.7 to 57.9% in urban watersheds. This range is narrower than reported for constructed wetlands. Mallin et al. (2002) reported a removal rate of -3.7% (an accrual). Mallin stated that this small increase is not statistically significant, and that the basin functioned well in nutrient removal overall. Greenway (2010) reported similar nitrogen removal efficiencies of 42.5% in wet weather and 57.9% in dry. Comings et al. (2000) also reported a narrow range of removal efficiencies for the overall treatment of 15 wet weather and 2 baseflow sampling conditions, 20-50%. Other studies consistently reported a narrow range as well.

Three studies reported total phosphorus removal rates ranging from 15.4 to 70% removal. These values were reported by Greenway (2010) for wet and dry weather, respectively. Studies by Mallin et al. (2002) and Vollersten et al. (2009) report overall removal efficiencies of 23 and 54.4%, respectively. As was the case for total nitrogen, total phosphorus removal efficiencies for retention basins lie in a relatively narrow range.

Overall, these studies demonstrate a relatively low degree of variability in terms of nutrient treatment performance among retention basins, usually ranging between 20 to 50% for both total nitrogen and phosphorus. The most influential factor in treatment capacity seemed to be flow regime (wet, dry, or average). Retention basins in urban watersheds only were reviewed, precluding a statement on the influence of watershed land use on treatment efficacy.

The studies reviewed pertaining to the effectiveness of retention basins for nutrient treatment are summarized in Table A-3.

**Table A-3: Summary of Studies Reviewed on Retention Basins Performance for Nutrient Treatment**

| Reference | Project Timeframe | Watershed Land Use | HRT (days) | Conditions | Influent Concentration (mg/L) | | Nutrient Treatment (%) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | TN | TP | TN | TP |
| Comings et al., 2000 | 5 months | Urban | | Overall | | | 20-50 | |
| Greenway, 2010 | 3 years | Urban | 8 in wetland 1 and 16-20 in wetlands 1 and 2 | Wet Weather | | | 42.5 | 70 |
| | | | | Dry Weather | | | 57.9 | 15.4 |
| Mallin et al., 2002 | 29 months | Urban | | Overall | 0.622 | 0.061 | -3.7 | 23.0 |
| Vollersten et al., 2009 | 1 year | Urban | | Overall | 3 | 0.4 | 24.5 | 54.4 |
| Weiss et al., 2007 | 20 years | Urban | | Overall | | | 52 | |

## Summary

The results of this literature review suggest that bioretention cells can achieve a higher level of nutrient treatment in storm water than constructed wetlands or retention basins. Studies of bioretention cells were completed in both laboratory and field settings, with laboratory results varying more widely than field results. A more thorough review of bioretention cells studies is warranted to determine their relative effectiveness to the other control measures reviewed in this memorandum. However, the bioretention cells in the studies reviewed here are smaller control measures than constructed wetlands or retention basins, so they are not necessarily interchangeable. The nutrient treatment capabilities of constructed wetlands or retention basins are more comparable to each other.

Several factors strongly influencing the effectiveness of bioretention cells, constructed wetlands, and retention basins include:

- Watershed land use characteristics
- Presence of direct sources of nutrients such as pipe inflow or waterfowl
- Flow regime (wet or dry weather)

Hydraulic retention time and influent concentration were not found to strongly influence the effectiveness of bioretention cells, constructed wetlands, and retention basins.

**DATE:** March 15, 2011

# MEMORANDUM

**FROM:** Maureen Habarth
Virginia Breidenbach
Scott Bell

**PROJECT:** KYSDCP1 – Task 5.2

**TO:** Jim Gibson, Sanitation District No. 1 of Northern Kentucky (SD1)

**CC:** Carrie Turner (LimnoTech)
Adrienne Nemura (LimnoTech)
Project File

**SUBJECT**: Bacteria Treatment Efficiency of Constructed Wetlands and Retention Basins.

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## Executive Summary


This memo presents a summary of findings on the treatment of bacteria from constructed wetlands and retention basins, based on a review of current technical literature. The goal of this literature review was to provide recommendations regarding bacteria treatment efficiency of retention basins and constructed wetlands, for use in future modeling, planning, and conceptual design work related to these watershed controls.

A total of 47 studies were reviewed, representing over 80 treatment wetland systems and 20 retention basin systems. A variety of wastewater sources were treated by these systems, including municipal wastewater with varying levels of treatment (untreated, primary, secondary, and tertiary treatment), storm water, and mixed sources. The most common constituent analyzed in the studies was fecal coliform bacteria, followed by total coliform bacteria, fecal streptococci, and *E. coli*. This memo focuses on results for fecal coliform.

For all of the studies, bacteria treatment efficiency was either reported or sufficient data was provided to calculate treatment efficiency, defined as the percent difference between the influent bacteria density and the effluent bacteria density. Treatment efficiencies varied widely overall with a range of 29 to 99% for wetlands and -15 to 98% for retention basins. The influence of hydraulic retention time (HRT) and influent density on fecal coliform treatment rates was also evaluated.

The reported bacteria treatment efficiencies are summarized in Table E-1, along with recommended ranges and median values for use in modeling and control measure performance calculations.

**Table E-1: Representative Fecal Coliform Treatment Efficiencies for Storm Water in Wetlands and Retention Basins**

| Statistic | Fecal Coliform Treatment (%) | |
|---|---|---|
| | Wetlands | Retention Basins |
| Actual Range | 54 to 89 | 57 to 79 |
| Recommended Range* | 55 to 90 | 55 to 80 |
| Actual Median | 66 | 69 |
| Recommended Median** | 65 | 65 |

*Range presented is the interquartile range (25th to 75th percentile) rounded to the nearest 5%.
**The recommended median represents a conservative rounding of actual medians down to the nearest 5% and accommodates the observation that reported median values for the two technologies are not statistically different than each other. It should be noted that a less conservative, but equally defensible, approach would be to round both medians up to 70%.

The lower end of the recommended range in Table E-1 represents the most conservative performance estimate based on the studies reviewed. It assumes a relatively underperforming or undersized control measure. The upper end of the range represents a control measured that is optimally sized, with excellent design, very good maintenance, and optimum operating conditions. The recommended median value represents well-designed control measures with good maintenance and average operating conditions.

General findings of the literature review are summarized below:

For wetlands:

- Percent treatment of fecal coliform is generally greater than 55% with a median treatment of 65% and a conservative upper treatment rate of 90%.
- The range of percent treatment was wider for storm water than other influent types (municipal wastewater receiving secondary or tertiary treatment).
- Lower treatment efficiencies are seen for lower influent densities.
- Percent treatment of fecal coliform is generally greater than 80% for influent densities >10,000 cfu/100mL.
- For HRT of 2 days or less, treatment rates vary from ~30% to 100%, whereas for HRT > 2 days, treatment rates are almost uniformly greater than 80% (with the exception of the two studies reported at 15 days HRT).
- For wetlands receiving only storm water, the median treatment rate was 66% with an interquartile range of 54% to 89%.

For retention basins:

- Percent treatment of fecal coliform is generally greater than 55% with a median treatment of 70% and a conservative upper treatment rate of 80%.
- Negative fecal coliform treatment rates were seen for two low influent densities; however other studies with similar influent densities reported treatment rates ranging from 55% to almost 100%.

- Few studies reported paired treatment efficiency and HRT or influent densities.
- The data evaluated in this study do not appear to support general conclusions regarding the relationship between bacteria treatment efficiency in retention basins and HRT. The dataset does not contain studies with HRT < 3 days; therefore, a conclusion can't be made about the effect of HRT < 3 days on bacteria treatment in retention basins.
- For retention basins receiving only storm water, the median treatment rate was 69% with an interquartile range of 57% to 79%.

## Overview

This memorandum presents the findings of a literature review of bacteria treatment rates for wetlands and retention basins, two types of Best Management Practices (BMPs) commonly used for storm water treatment. The focus of this review was to evaluate the effectiveness of these BMPs of removing bacteria from wastewater as reported in the literature, with particular consideration given to storm water studies. The results of this review could be used to guide the selection of bacteria treatment efficiencies for use in modeling or for use in BMP performance calculations.

The findings of this review are presented in the following sections:

Description of BMPs Examined

Overview of Studies Reviewed

Key Findings

Summary

References

Attachment A: Detailed Study Results

## Description of BMPs Examined

Bacterial treatment efficiency in treatment wetlands and retention basins was examined in this literature review. Basic descriptions of these two treatment methods are provided below.

### *Treatment Wetlands*

The use of constructed or natural wetlands to treat water pollution has been well studied and is reported extensively in the peer reviewed literature. Although the creation of treatment wetlands can be expensive, these systems can be a much more economical option in comparison to traditional chemical and physical treatment, especially for smaller municipalities (Mitsch and Gosselink, 2007).

There are three primary types of treatment wetlands defined by the flow of water through the wetland: natural, surface flow, and subsurface flow. Variations on these primary types of treatment wetlands include horizontal flow and vertical flow. By choosing the type of treatment wetland or hybrid wetland system appropriate for the application, it is possible to achieve a high level of treatment for a variety of pollutants including bacteria, nutrients, metals, and solids. Treatment wetlands can also provide additional environmental and community benefits, including habitat creation, water level control, aesthetic enhancement, and public education opportunities.

### *Retention Basins*

While detention basins (dry ponds) are designed to be dry and temporarily hold high-peak-flow runoff, retention basins (wet ponds) are designed to have a permanent pool of water (Novotny, 2003).

Retention control measures provide both retention and treatment of water (USEPA, 1999). Traditionally, basins/ponds have been utilized primarily for storm water control with less consideration given to potential water quality benefits. However, with proper siting and design, retention basins have been proven to effectively reduce bacteria and other pollutants while reducing peak flows.

## Overview of Studies Reviewed

A literature search was conducted in order to summarize the performance of treatment wetlands and retention basins for bacteria treatment. Numerous studies have reviewed the potential for these systems to remove bacteria from wastewater associated with municipal, agricultural, and septic discharges; the use of treatment wetlands and retention basins for removing bacteria from storm water is also gaining increasing presence in the literature.

A total of 47 studies were reviewed, representing over 80 treatment wetland systems and 20 retention basin systems. The majority of studies were conducted in either North America or Europe. Of the 23 North American studies reviewed, 19 were conducted in the United States, three in Canada, and one in Honduras. In Europe, 13 studies were generally evenly distributed among 10 different countries. Table 2 summarizes the geographic locations of the reviewed studies by continent.

**Table 2**: **Count of studies reviewed by continent of study location**

| Continent | # Studies |
|---|---:|
| Africa | 3 |
| Asia | 2 |
| Australia | 4 |
| Europe | 13 |
| North America | 23 |
| South America | 0 |
| Multiple continents | 2 |
| Total | 47 |

Fecal coliform bacteria was the most common bacteria constituent analyzed in the reviewed studies, followed by total coliform bacteria, fecal streptococci, and *E. coli*. Table 3 summarizes the type of bacteria analyzed and the number of studies reporting results for each type. Some of the studies reviewed reported treatment efficiencies for additional bacterial species not listed below; those results are not included in this review. Note that many studies analyzed the treatment of multiple bacteria types.

**Table 3**: **Studies reviewed by type of bacteria**

| Bacteria Analyzed | # Studies |
|---|---|
| *E. coli* | 7 |
| Fecal coliform | 36 |
| Fecal streptococci | 12 |
| Total coliform | 17 |
| Multiple groups (averaged) | 3 |

The type of influent treated varied considerably among the studies, with domestic or municipal wastewater being the most commonly reported source, followed by storm water. For simplicity, domestic and municipal wastewater sources are combined in the category "municipal" in this memorandum. Table 4 summarizes the studies reviewed by wastewater source. Note that many studies analyzed the treatment of multiple wastewater sources.

**Table 4: Studies reviewed by wastewater source**

| Wastewater Source | # Studies |
|---|---|
| Municipal, raw | 19 |
| Municipal, secondary effluent | 6 |
| Municipal, tertiary effluent | 2 |
| Storm water | 10 |
| Combination of sources/other | 12 |

Bacteria treatment efficiencies were reported or calculated for all 47 studies. Influent densities and hydraulic retention times were reported for only some of the treatment systems investigated, as shown in Table 5. Note that many studies analyzed more than one treatment system such that multiple influent densities and HRTs may be reported within a single study.

**Table 5: Studies reviewed reporting influent density and hydraulic retention time (HRT)**

| | Influent Density # Studies | HRT # Studies |
|---|---|---|
| Wetlands: | | |
| Municipal, raw | 16 | 12 |
| Municipal, secondary | 5 | 4 |
| Municipal, tertiary | 3 | 2 |
| Storm water | 5 | 2 |
| Mixed/other | 7 | 4 |
| Retention basins: | | |
| Municipal, raw | 1 | 2 |
| Municipal, secondary | 0 | 0 |
| Municipal, tertiary | 0 | 0 |
| Storm water | 7 | 4 |
| Mixed/other | 0 | 0 |

## Key Findings

This section presents key findings regarding bacterial treatment from wetlands and retention basins based on the review of treatment rates reported in the literature. More detail on individual study results, including descriptions of the various conditions impacting bacteria treatment in those studies, is provided in Attachment A.

The findings are presented in the following subsections:

Range of Reported Bacteria Treatment Efficiencies

Effect of Hydraulic Retention Time

Effect of Influent Density

Storm Water-Specific Results

### *Range of Reported Bacteria Treatment Efficiencies*

A wide range of bacteria treatment efficiencies was reported for both wetland and retention basin treatment systems. Treatment rates were influenced by numerous design parameters and influent conditions. (Individual study results and influencing factors are described in more detail in Attachment A.)

The studies reviewed on wetlands are generally in agreement that constructed treatment wetlands can achieve as high as virtually 100% treatment of bacteria and typically achieve bacteria treatments in the 90 to 99 percent range, although these rates depend on system design, operating conditions, and environmental factors. The overall range of bacteria treatment efficiencies in wetlands reported was 26 to 100% for fecal coliform and 33% to 100% for *E. coli*.

Fewer studies were available in the literature that evaluated bacteria treatment in retention basins. Fecal coliform treatment in retention basins ranged from -15 (indicating an increase in density) to 99%. A single study reported an *E. coli* treatment rate of 46% in a retention basin system.

In order to be useful for selecting appropriate treatment efficiencies for modeling applications or performance calculations, a better understanding of the factors influencing bacteria treatment is necessary. Several studies reported increased bacteria treatment with increased hydraulic retention time (HRT). Therefore, this relationship was evaluated for studies where HRT was reported. The influence of influent density on treatment rates was also investigated, given the wide range in wastewater sources treated by BMPs in the literature reviewed.

### *Effect of Hydraulic Retention Time*

Several of the individual studies reviewed reported differing treatment efficiencies with varying HRTs. Therefore, the relationship between HRT and bacteria treatment rates was investigated for the studies reporting both parameters (for fecal coliform: 35 cases for wetlands and 10 cases for retention basins). For wetlands, the results indicate that bacteria treatment rates of greater than 80% are achieved in almost all cases with HRT > 2 days, as shown in Figure 1. Treatment rates vary widely for HRTs < 2 days. For retention basins, the influence of HRT is not clearly seen (Figure 2). However, no results were available in the dataset for HRT < 3 days.

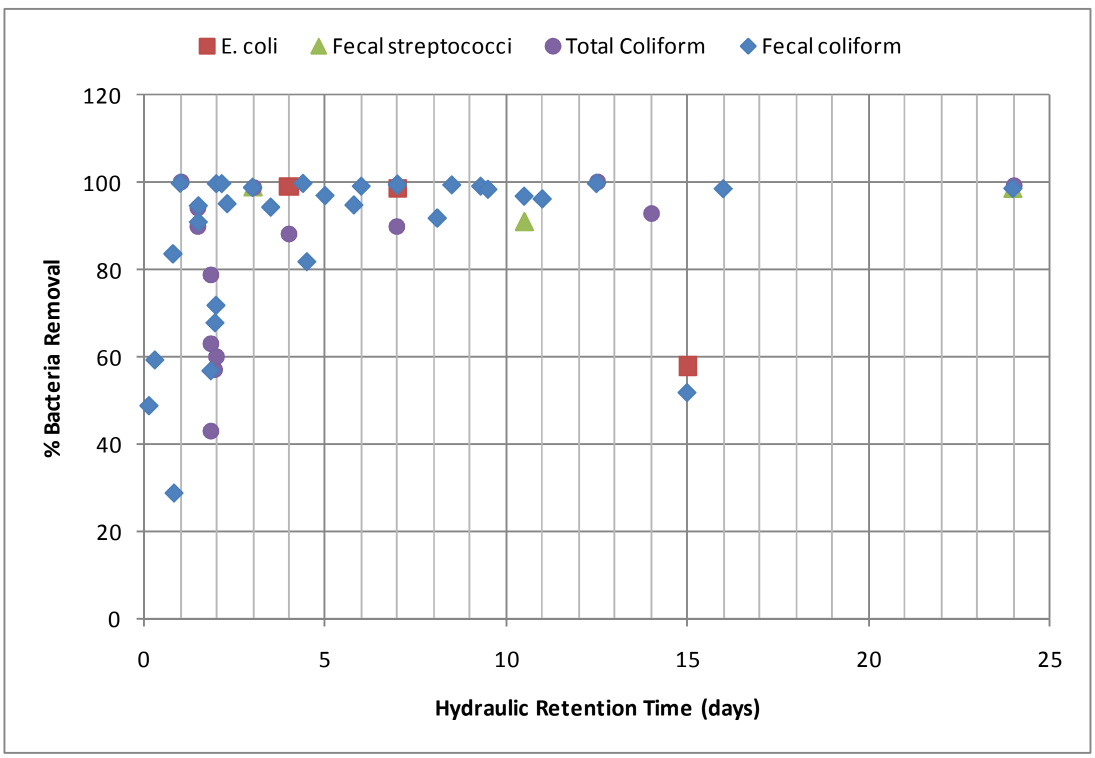


**Figure 1: Influence of Hydraulic Retention Time on Bacteria Treatment in Wetlands**

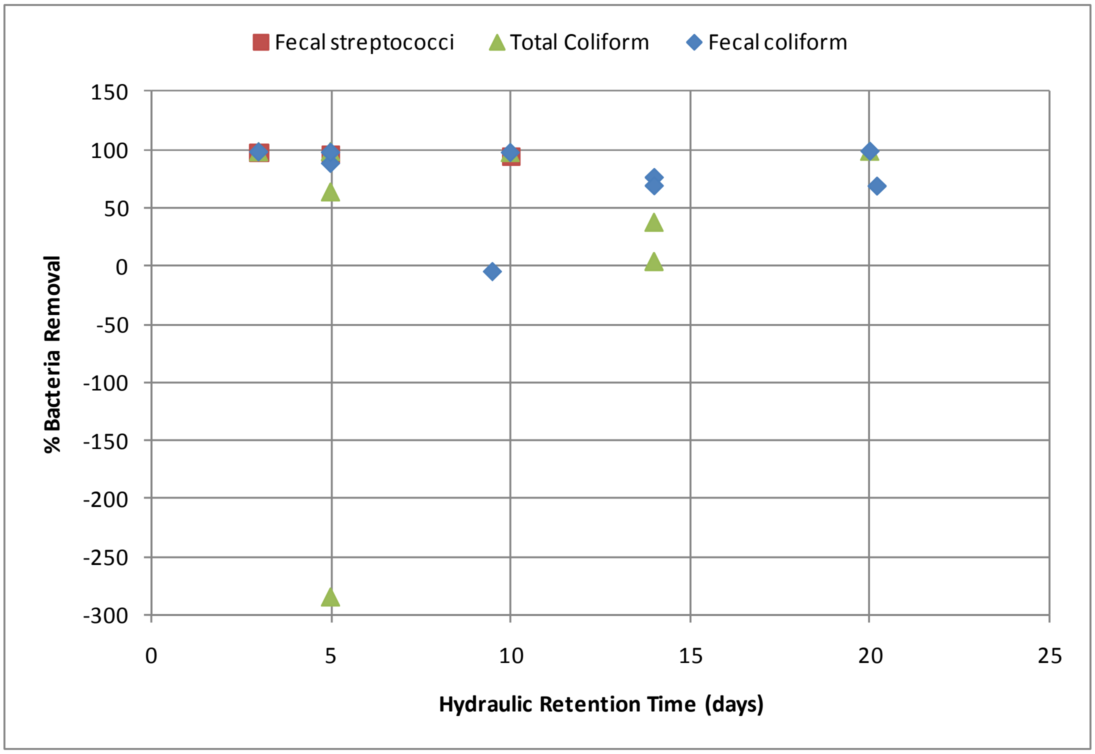


**Figure 2: Influence of Hydraulic Retention Time on Bacteria Treatment in Retention Basins**

### *Effect of Influent Density*

Comparing bacteria treatment rates reported as percent differences in influent and effluent densities does not always provide the most accurate understanding of a BMP's effectiveness. A higher influent density may result in a greater percentage treatment, while a lower influent density in the same system may result in a lower percentage treatment with effluent densities being similar in both instances. Where paired influent density and percent treatment rates were reported, the influence of influent density on reported treatment rates was investigated (for fecal coliform: 65 cases for wetlands and 14 cases for retention basins).

For wetlands, treatment rates of greater than 80% are generally achieved with influent densities greater than 10,000 cfu/100 mL, as shown in Figure 4. Treatment efficiency varies widely (29 to 99%) for influent densities less than 10,000 cfu/100mL.

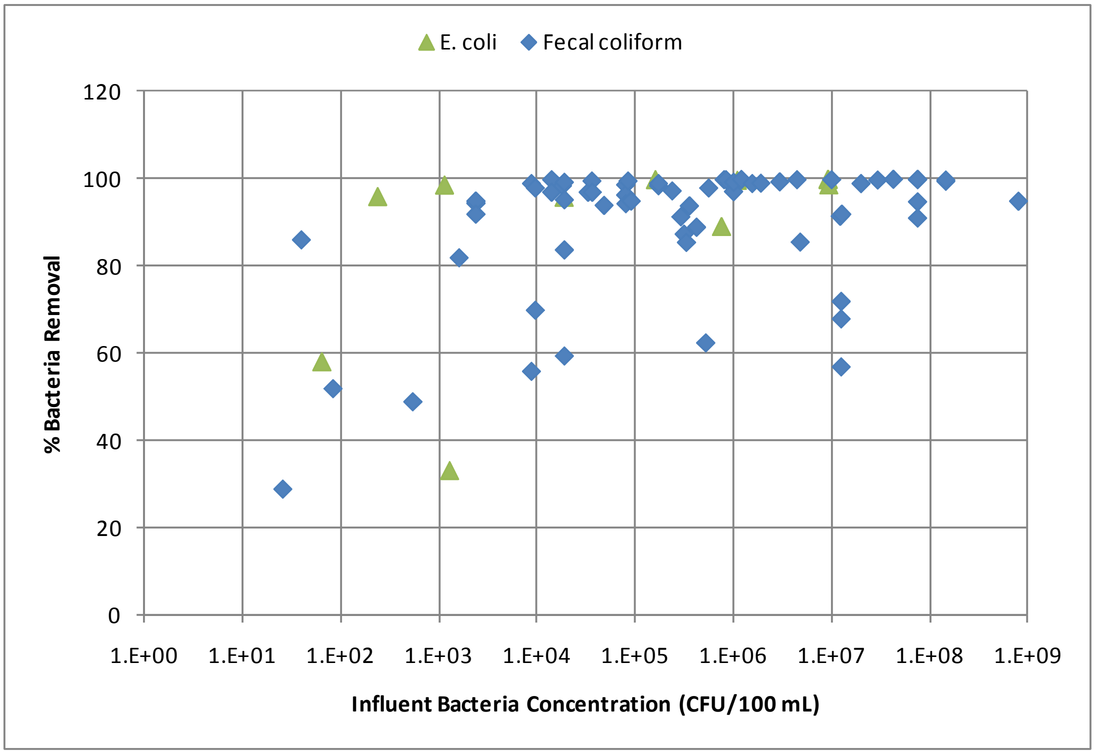


**Figure 3: Influence of Influent Density of Bacterial Treatment in Wetlands**

For retention basins, the relationship between influent density and treatment efficiency is unclear, given the limited data available as shown in Figure 4.

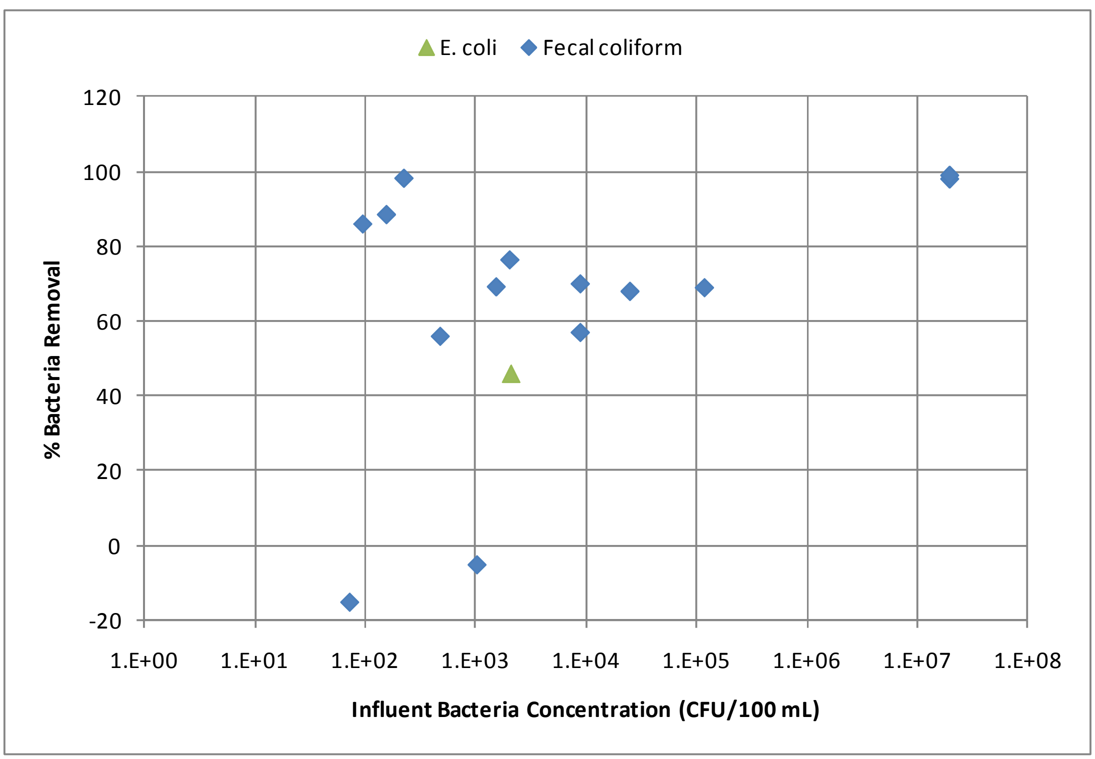


**Figure 4: Influence of Influent Density of Bacterial Treatment in Retention Basins**

Figure 5[1] shows the fecal coliform influent densities presented in Figure 3 by influent type. The bacterial treatment efficiencies associated with these influent densities are presented in Figure 6. Storm water influent densities are lower in general than those for most other wastewater sources, with the lowest mean, minimum, and interquartile (25th to 75th percentile) range. A wider range of treatment rates and the lowest median treatment rate are associated with storm water.

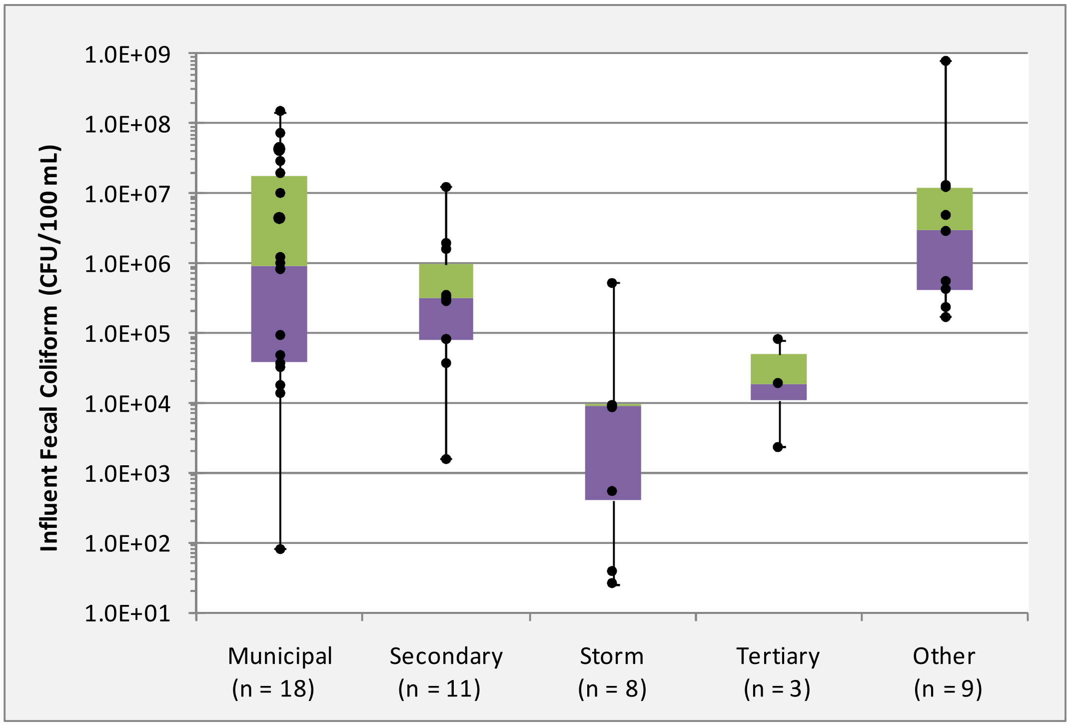


**Figure 5: Influent Fecal Coliform Densities for Wetlands**

[1] Box and whisker plots indicate the minimum (lower line endpoint), maximum (upper line endpoint), 25th percentile (lower edge of lower colored bar), median (line where bar colors change), and 75th percentile (upper edge of upper colored bar) of the data.

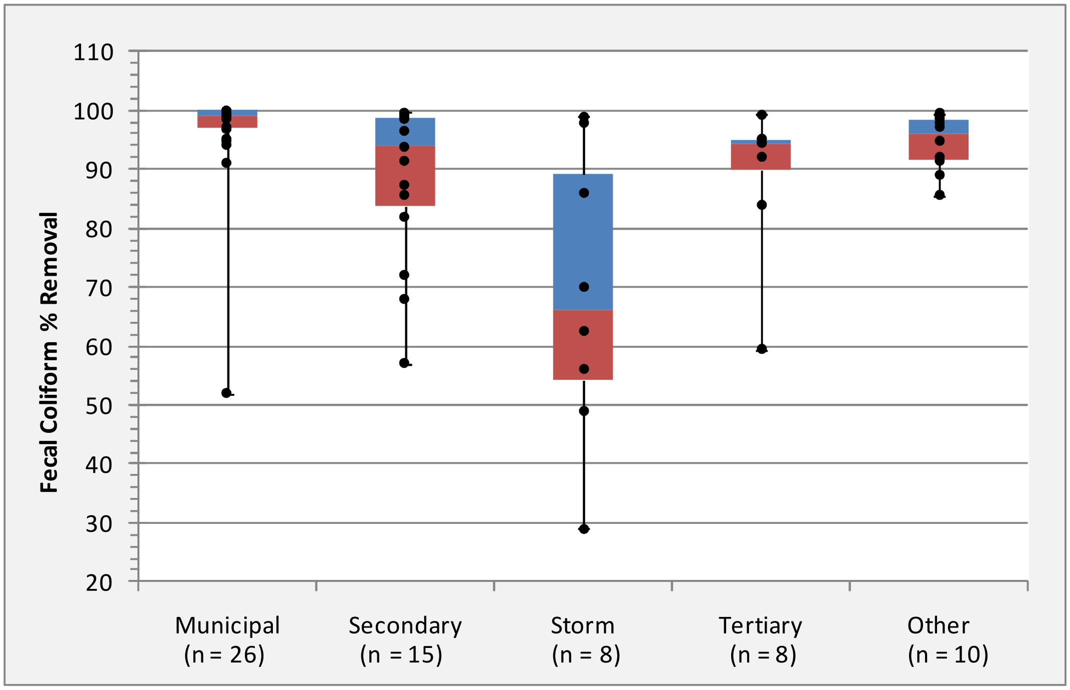


**Figure 6: Fecal Coliform Treatment Efficiencies for Wetlands**

### *Storm Water-Specific Results*

This section summarizes influent densities and treatment efficiencies reported for studies of wetlands and retention basins where storm water was the influent. Results are presented for fecal coliform.

Influent densities of fecal coliform in storm water were generally similar in the wetland and retention basin studies, as indicated in Figure 7. As seen in Figure 8, fecal coliform treatment rates were also similar. However, a wider range of treatment efficiencies is reported for retention basins than for wetlands. Table 6 presents the densities and treatment rates associated with these two figures. While the median treatment rate is slightly higher for retention basins than wetlands (57% vs. 54%, respectively), the upper end of the interquartile range is approximately 10% greater for wetland treatment systems than that for retention basins.

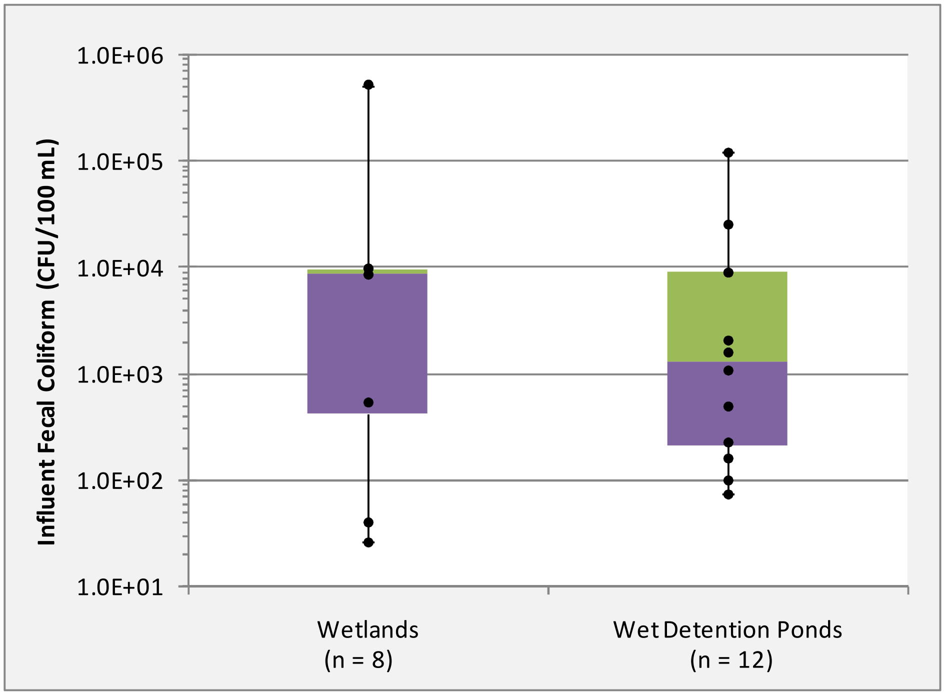


**Figure 7: Influent Fecal Coliform Densities in Storm Water**

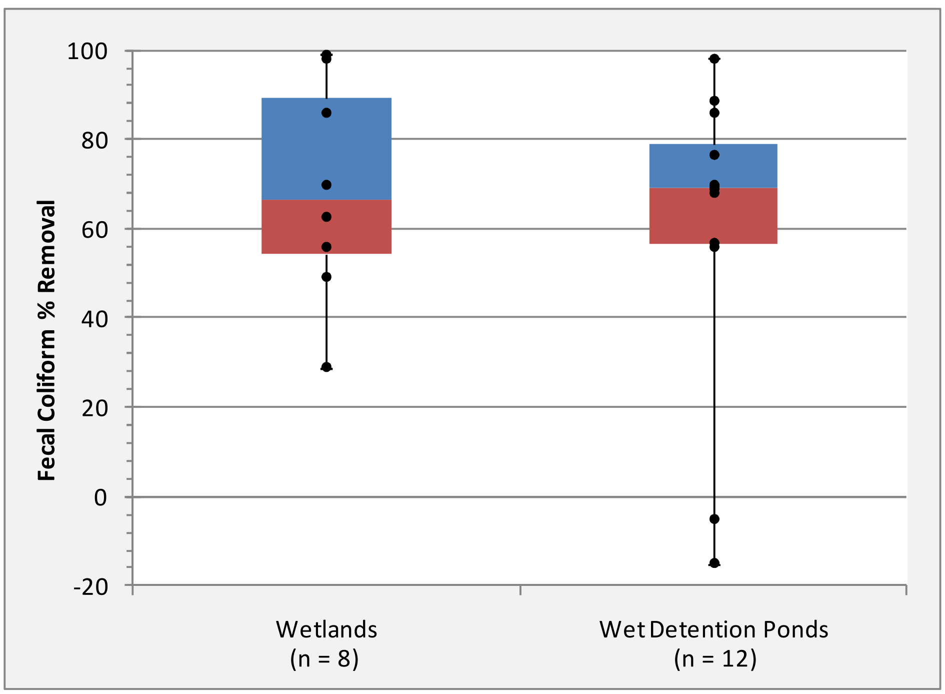


**Figure 8: Fecal Coliform Treatment from Storm Water Influent**

**Table 6: Influent Fecal Coliform Densities and Percent Treatment for Storm Water**

| Statistic | Influent Fecal Coliform Density (CFU/100mL) | | Fecal Coliform Treatment (%) | |
|---|---|---|---|---|
| | Wetlands | Retention Basins | Wetlands | Retention Basins |
| Min | 2.60E+01 | 7.40E+01 | 29 | -15 |
| Max | 5.21E+05 | 1.20E+05 | 99 | 98 |
| Median | 8.72E+03 | 1.31E+03 | 66 | 69 |
| 25th Percentile | 4.15E+02 | 2.12E+02 | 54 | 57 |
| 75th Percentile | 9.56E+03 | 9.03E+03 | 89 | 79 |
| Number | 8 | 12 | 8 | 12 |

Because of the similarity of the median values depicted in Figure 8, analyses were conducted to determine whether the two data sets are statistical different. A series of statistical tests were conducted using the Minitab software package to determine if observed treatment efficiencies from wetland systems were significantly different than treatment efficiencies from retention basins. A non-parametric test (the Kruskal–Wallis one-way analysis of variance by ranks) was applied and did not show a significant difference in median treatment efficiency at a significance level of $p=0.10$. In addition, a parametric two-sample t-test was performed, which also did not show a significant difference in average treatment efficiency at a $p=0.10$ significance level. These results indicate that the median treatment efficiency for wetlands and retention basins are not statistically different.

The interquartile range has been used by others (Schueler, et al., 2007) to describe the range of treatment efficiencies considered to be appropriate for use when evaluating variations in pollutant treatment expected from BMPs. Properly designed and maintained BMPs might be given a higher value in this range, while underperforming or undersized BMPs might be given a lower value in this range.

Based on the analyses and review of study results presented in this and preceding sections, a range of representative fecal coliform treatment efficiencies was selected for each BMP type for performance calculations and modeling purposes based on the interquartile results, as indicated in Table 7.

**Table 7: Recommended Fecal Coliform Treatment Efficiencies for Storm Water in Wetlands and Retention Basins**

| Statistic | Fecal Coliform Treatment (%) | |
|---|---|---|
| | Wetlands | Retention Basins |
| Actual Range | 54 to 89 | 57 to 79 |
| Recommended Range* | 55 to 90 | 55 to 80 |
| Actual Median | 66 | 69 |
| Recommended Median** | 65 | 65 |

*Range presented is the interquartile range (25th to 75th percentile) rounded to the nearest 5%.
**The recommended median represents a conservative rounding of actual medians down to the nearest 5% and accommodates the observation that reported median values for the two technologies are not statistically different than each other. It should be noted that a less conservative, but equally defensible, approach would be to round both medians up to 70%.

The lower end of the recommended range in Table 7 represents the most conservative performance estimate based on the studies reviewed. It is not the absolute worst case, but assumes a relatively underperforming or undersized control measure. The upper end of the range represents a control measured that is optimally sized, with excellent design, very good maintenance, and optimum operating conditions. The recommended median value represents well-designed control measures with good maintenance and average operating conditions.

## Summary

Bacteria treatment in wetlands and retention basins was investigated through a review of available literature. A total of 47 studies were reviewed, representing over 80 treatment wetland systems and 20 retention basin systems. Treatment efficiencies, defined as the percent difference between the influent bacteria density and the effluent bacteria density, were reported or calculated for all of the studies. Treatment efficiencies ranged widely overall with a range of 29 to 99% for wetlands and -15 to 98% for retention basins.

Where data were available, the influence of hydraulic retention time and influent density on bacteria treatment was evaluated. Based on the analyses, a range of representative fecal coliform treatment efficiencies (55 to 90% for wetlands and 55 to 80% for retention basins) was selected for each control measure type for design and modeling purposes. Both control measure types have a recommended median treatment efficiency of 70%. The following statements can be made regarding bacteria treatment in wetlands and retention basins:

For wetlands,

- Percent treatment of fecal coliform is generally greater than 55% with a median treatment of 65% and a conservative upper treatment rate of 90%.
- The range of percent treatment was wider for storm water than other influent types (municipal wastewater receiving secondary or tertiary treatment).
- Lower treatment efficiencies are seen for lower influent densities.
- Percent treatment of fecal coliform is generally greater than 80% for influent densities >10,000 cfu/100mL.
- Increasing hydraulic retention time increases percent treatment : HRT >2 days results in treatment efficiencies of 80% or greater.

For retention basins,

- Percent treatment of fecal coliform is generally greater than 55% with a median treatment 70% and a conservative upper treatment rate of 80%.
- Negative fecal coliform treatment rates were seen for two low influent densities; however other studies with similar influent densities reported treatment rates ranging from 55% to almost 100%.
- Few studies reported paired treatment efficiency and HRT or influent densities.
- The dataset does not contain studies with HRT < 3 days; therefore, a conclusion can't be made about the effect of HRT on bacteria treatment in retention basins.

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# Attachment A: Detailed Study Results

## Summary of Individual Bacteria Treatment Study Results

Treatment wetland and retention basin performance in removing bacteria from wastewater or storm water is dependent on many factors, as evidenced by the studies reviewed. The focus of the individual studies varied widely. For example, some studies examined the performance of vegetated systems versus unvegetated or differences in performance among different species of vegetation. Some studies examined the effects of hydraulic retention time, loading rate, or influent flow rate, while others focused on performance differences between substrate types (i.e. sand versus clay). Many studies reported on the influence of temperature or sunlight on bacteria survival, with some focusing particularly on seasonal differences in wetland performance, especially in cold climates.

Bacteria treatment efficiency was typically reported as an average percent treatment over the monitoring period and was calculated based on the difference in bacteria density measured at the inlet and outlet of the pond or wetland system. Following is a discussion of study results for treatment wetlands and retention basins.

### *Treatment Wetlands*

The studies reviewed on wetlands are generally in agreement that constructed treatment wetlands can achieve as high as virtually 100% treatment of bacteria and typically achieve bacteria treatments in the 90 to 99 percent range, although these estimations depend strongly on system design, operating conditions, and environmental factors.

In general, the study results suggest that wetlands are typically effective at reducing densities of fecal coliform (FC) bacteria. Two of the lowest reported percent treatments for FC were 29 and 49 percent, achieved by natural palustrine wetlands treating storm water runoff (Reinelt and Horner, 1995). Birch et al. (2004) also reported a low 26 percent treatment of FC from storm water during a single high flow event, although three smaller events achieved FC treatments of 83, 98, and 99 percent. Cameron et al. (2003) reported a low 52 percent treatment of FC bacteria for a constructed wetland receiving municipal lagoon effluent, and Kaseva (2004) reported a range of 57-72 percent FC treatment in three constructed wetlands receiving secondary effluent. Hathaway et al. (2009) reported 56 percent FC treatment for a constructed storm water wetland draining a mostly residential area. Toet et al. (2005) reported 59.5 percent treatment of FC for a surface flow constructed wetland with an HRT of 0.3 days; however, when HRT was increased to 0.8, 2.3, and 9.3 days, treatment rates increased to 83.8, 95.3, and 99.3 percent, respectively. Excluding these studies, FC treatment rates by constructed wetlands were typically in the range of 90 to 100 percent, with just a couple of studies reporting treatment efficiencies in the 70 or 80 percent range (Hathaway et al., 2008, Ontkean et al., 2003 and Vymazal, 2005b).

Results for the seven studies reporting *E. coli* bacteria treatment suggest that treatment of *E. coli* by constructed wetlands can vary significantly. In five of the studies reviewed, treatment rates of 96 percent or greater were achieved. However, Cameron et al. (2003) reported a low treatment rate of 58 percent by a constructed wetland receiving municipal lagoon effluent. Additionally, Hathaway et al. (2009) reported only 33 percent treatment of *E. coli* by a constructed wetland receiving storm water runoff from a mostly residential area. A study completed for the City of Toronto (SWAMP, 2005) found a large constructed wetland to be 75 percent effective at removing *E. coli* from storm water.

Twelve articles on treatment wetlands examined fecal streptococci (FS) treatment, with percent treatment ranging from 0 to 100 percent. Vrhovšek et al. (1996) found a constructed wetland to remove 0 percent of FS bacteria during one of three monitoring dates; however, the same system removed 96 percent and 99.9 percent of FS during the other two dates. The zero percent treatment was considered to be anomalous. Among the 11 other studies incorporating FS, nine studies reported treatment rates in the range of 84 to 100 percent, with two studies reporting treatment rates in the 70 percent range (Bavor et al., 2001, Lau and Chu, 2000). Based on the majority of studies on the treatment of FS bacteria by constructed wetlands, these systems seem to be very effective for the treatment of this class of bacteria.

Sixteen articles regarding treatment wetland performance for total coliform bacteria are generally in consensus that these systems are typically very effective at removing total coliform bacteria. A few exceptions are as follows. Diemont (2006) reported a range of 39-87 percent treatment by free water surface wetlands in Honduras treating municipal wastewater. The low treatment rate of 39 percent was achieved in just one of many tests, with other regimes and monitoring periods removing 62-87 percent of total coliform. Kaseva (2004) reported treatment rates of 43, 57, and 60 percent of total coliform bacteria for three constructed wetland units treating secondary effluent. An unplanted control provided the lowest treatment efficiency (43 percent). Vymazal (2005b) reported total coliform treatment from a database of 60 constructed wetlands, with an average treatment of 65.1 percent for free water surface constructed wetlands. The majority of studies on the treatment of total coliform by constructed wetlands reported results in the range of 80 to greater than 99 percent.

The studies reviewed pertaining to the effectiveness of treatment wetlands for bacteria treatment are summarized in Table A-1.

**Table A-1: Summary of Studies Reviewed on Treatment Wetland Performance for Bacteria Treatment**

| Reference | Wastewater Source | HRT (days) | Influent Density (CFU/100 mL) | | Pollutant Treatment (%) | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Fecal coliform | *E. coli* | Fecal coliform | *E. coli* | Fecal streptococci | Total coliform |
| Addo et al., 2006 | Domestic | Not provided | 5.4-1,600 x $10^3$ | 2.6-320 x $10^3$ | > 99.9 | > 99.9 | -- | |
| Ansola et al., 2003 | Municipal | 1.6-2.7 | 14,034 | | > 99.9 | | | |
| | | 10.5 | | | 95-99 | | 89-93 | |
| Antonious et al., 1996 | Domestic (single-family) | 4.39 | 1.2 x $10^6$ | | 99.93 | | | |
| Arias et al., 2003 | Municipal | Not provided | 3-56 x $10^6$ | | >99.8 | | 99.1-99.9 | >99.9 |
| Bavor et al., 2001 | Storm water | Not provided | | | | | 79 | |
| Birch et al., 2004 | Storm water | Not provided | 18,211-1,022,851 | | 26-99 | | | |
| Cameron et al., 2003 | Municipal | 15 | 82.77 | 64.85 | 52 | 58 | | |

| Reference | Wastewater Source | HRT (days) | Influent Density (CFU/100 mL) | | Pollutant Treatment (%) | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Fecal coliform | *E. coli* | Fecal coliform | *E. coli* | Fecal streptococci | Total coliform |
| Davies et al., 2000 | Storm water | Not provided | | | | | | 79-87 |
| Decamp et al., 2001 | Domestic | Not provided | $3.3\text{-}11 \times 10^8$ | | 67 | | | |
| | | | | | 15-39 | | | |
| Diemont, 2006 | Municipal | 1.1-2.6 | $230\text{-}590 \times 10^6$ | | | | | 39-87 |
| | | | | | | | | 79 |
| Garcia et al., 1997 | Domestic (rural) | 24 | Not provided | | 98.80 | | 98.67 | 99.48 |
| Garcia et al., 2008 | Domestic (rural) | 3 | 4.36-7.6 log CFU/100 mL | | 99 | | 99 | 99 |
| | | 3 | | | >99 | | 99 | >99 |
| Ghermandi et al., 2007 | Secondary effluent | Not provided | $1.9 \times 10^6$ | $9.4 \times 10^6$ | 99.1 | 98.7 | | 91.4 |
| | Tertiary effluent | | $8 \times 10^4$ | $1.9 \times 10^4$ | 94.4 | 95.8 | | 95.8 |
| | Agricultural | | $4.2 \times 10^5$ | $7.6 \times 10^5$ | 89 | 89.1 | | 86 |
| Greenway, 2005 | Secondary effluent | 16 | 79,500 | | 98.7 | | | |
| | | 11 | | | 96.4 | | | |
| | | 7-10 | 84,000 | | 99.6 | | | |
| | | 7 | 36,000 | | 99.6 | | | |
| | | 4-5 | 1,600 | | 82 | | | |
| | Primary effluent | 2 | $1 \times 10^7$ | | 99.9 | | | |
| Hathaway et al., 2008 | Storm water | Not provided | 9,560 | | 70 | | | |
| | | | 8,724 | | 99 | | | |
| Hathaway et al., 2009 | Storm water (residential) | Not provided | 9,560 | 2,400 | 98 | 96 | | |
| | | | 8,724 | 1,295 | 56 | 33 | | |
| Hench et al., 2003 | Domestic (rural) | 6-8 | 7.6-8.4 log CFU/100 mL | | 99.5 | | | |
| | | | | | 99.8 | | | |
| Jillson et al., 2000, 2001 | Domestic (community) | 5 | $10^5\text{-}10^7$ | | 96.3, 98 | | | |
| | Domestic (single-family) | 6 | | | 99.3 | | | |
| Karathanasis et al., 2003 | Domestic (single-family) | Not provided | $90.4 \times 10^3$ | | 95 | | 93 | |
| | | | $36.4 \times 10^3$ | | 97 | | 98 | |
| | | | $32.9 \times 10^3$ | | 97 | | 94 | |
| | | | $48 \times 10^3$ | | 94 | | 94 | |

| Reference | Wastewater Source | HRT (days) | Influent Density (CFU/100 mL) | | Pollutant Treatment (%) | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Fecal coliform | *E. coli* | Fecal coliform | *E. coli* | Fecal streptococci | Total coliform |
| Karim et al., 2004 | Secondary effluent | 5-14 | Not provided | | 98.6 | | | |
| Kaseva, 2004 | Secondary effluent | 1.85 | 8-17 x $10^6$ | | 57 | | | 43 |
| | | 1.96 | | | 68 | | | 57 |
| | | 1.99 | | | 72 | | | 60 |
| Keffala et al., 2005 | Domestic | Not provided | 8.35 x $10^5$ | 9.24 x $10^6$ | >99.9 | >99.99 | | >99.9 |
| | | | 4.43 x $10^6$ | 9.24 x $10^6$ | >99.9 | >99.9 | | >99.9 |
| Knowlton et al., 2002 | Mixed primary and secondary effluent | 2 | 237,077 | | 97.3 | | 98.5 | |
| Lau et al., 2000 | Domestic, agricultural, industrial | 8 | 172 x $10^3$ | | 100, 97 | | 87, 72 | |
| | | | | | 99, 99 | | 100, 99 | |
| Mantovi et al., 2003 | Domestic, Dairy | 10 | | 1.1 x $10^6$ | | 99.7 | 98.8 | 99.6 |
| Maschinski et al., 1999 | Secondary effluent | Not provided | 1.72 x $10^3$-3.1 x $10^6$ | | >99 | | | >99 |
| Mbuligwe, 2005 | Domestic | 1 | 42.6 x $10^6$ | | 99.99 | | | 99.99 |
| Newman et al., 2000 | Dairy | 41 days (average) | 557,378 | | 98 | | | |
| Ontkean et al., 2003 | Agricultural runoff | Not provided | 12-67 | | 73.2-99 | | | |
| Perkins et al., 2000 | Secondary effluent | Not provided | 17,700-562,000 | | 91.4 | | 86.1 | |
| | | | 37,300-625,000 | | 85.5 | | 82.5 | |
| | | | 20,900-604,800 | | 87.4 | | 88.7 | |
| | | | 33,100-676,800 | | 93.9 | | 90.3 | |
| Quiñónez-Díaz et al., 2001 | Municipal | 1-2 | 7.5 x $10^7$ | | 94.8 | | | 94 |
| | | 1-2 | | | 91.1 | | | 89.8 |
| | | 10-15 | | | 99.9 | | | 99.97 |
| | | 10-15 | | | 99.99 | | | 99.99 |
| Reinelt et al., 1995 | Storm water runoff | 3.3 hrs | 441-746 | | 49 | | | |
| | | 20 hrs | 20-31 | | 29 | | | |

| Reference | Wastewater Source | HRT (days) | Influent Density (CFU/100 mL) | | Pollutant Treatment (%) | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Fecal coliform | *E. coli* | Fecal coliform | *E. coli* | Fecal streptococci | Total coliform |
| Steer et al., 2005 | Domestic (single-family) | Not provided | 4.17-4.33 log CFU/100 mL | | 98.5 | | | |
| Stenström et al., 2001 | Municipal | 7 | | $4 \times 10^2$-$1.9 \times 10^3$ | | 99.8, >97.5 | | 87.9, 92 |
| | Storm water | 3-5 | | Not provided | | 99 | | 88 |
| SWAMP, 2005 | Storm water | Not provided | | Not provided | | 75 | | |
| Toet et al., 2005 | Tertiary effluent | 0.3 | $8.8$-$28.9 \times 10^3$ | | 59.5 | | | |
| | | 0.8 | | | 83.8 | | | |
| | | 2.3 | | | 95.3 | | | |
| | | 9.3 | | | 99.3 | | | |
| Tunçsiper, 2007 | Tertiary effluent | 1.6-5.3 | 2,077-2,659 | | 95, 94 | | | |
| | | 2.7-8.8 | | | 95, 95 | | | |
| | | 3.8-12.4 | | | 91, 93 | | | |
| Vacca et al., 2005 | Domestic | Not provided | | | | | | 99.5 |
| | | | | | | | | 99.7 |
| | | | | | | | | 96 |
| | | | | | | | | 98 |
| | | | | | | | | 99.2 |
| | | | | | | | | 99.7 |
| Vrhovšek et al., 1996 | Food processing wastewater | Not provided | $8 \times 10^5$-$1.6 \times 10^9$ | | 90-99.9 | | 0-99.9 | 99-99.9 |
| Vymazal, 2005 (1) | N/A | Not provided | $1.27 \times 10^7$ | | 92 | | | |
| Vymazal, 2005 (2) | N/A | Not provided | $1.22 \times 10^7$ | | 91.5 | | 92.6 | 88.1 |
| | | | $4.77 \times 10^6$ | | 85.6 | | 84 | 65.1 |
| | | | $2.96 \times 10^6$ | | 99.4 | | 97.7 | 99.1 |
| Zdragas et al., 2002 | Secondary effluent | 14 | | | | | | 85.78-100 |

## *Retention Basins*

Eleven of the studies reviewed focused on the use of retention basins for the treatment of bacteria from storm water or wastewater. Reported treatment efficiencies vary widely, suggesting bacteria treatment in these systems is sensitive to design, operation, environmental conditions, and other factors and/or that additional research is needed to further characterize the capability of retention basins to remove bacteria.

Of the eleven studies, only two examined the use of retention basins to treat domestic wastewater, while all of the other studies involved storm water retention basins. García et al. (1997) determined that an algal pond and a waste stabilization pond could remove greater than 98 percent of fecal and total coliform and greater than 95 percent of fecal streptococci from rural domestic wastewater in a rural setting. García et al. (2008) studied similar systems and again reported greater than 98 percent treatment of FC and TC and 94 percent treatment of FS from domestic wastewater.

Among nine studies reporting treatment of fecal coliform bacteria by retention basins, treatment efficiencies range from -15 percent (a 15 percent increase in density between the inlet and outlet) and greater than 99 percent. Mallin et al. (2002) monitored three storm water retention ponds over a 29 month period. One pond increased FC densities by 15 percent, while the other two decreased FC by 56 and 86 percent. This was believed to be due in part to improper design, in particular an insufficient length-to-width ratio. Borden et al. (1998) studied two storm water ponds draining similar watershed areas. One pond, draining largely dairy farm and wooded areas, removed 48 to 90 percent of FC over the monitoring period. The second pond, draining a petroleum tank farm and commercial and forested areas, increased FC densities by 5 percent.

Only one study reported on the efficiency of a retention basin for removing *E. coli* from storm water; Hathaway et al., (2009) estimated a 46 percent treatment rate of *E. coli* by a wet pond in North Carolina.

Three studies examined the treatment of fecal streptococci bacteria by retention basins. As described above, García et al. (1997) and García et al. (2008) found that a high-rate algal pond could achieve between 94 and 96 percent treatment of FS, and a waste stabilization pond could remove greater than 97 percent of FS from domestic wastewater.

Four studies reported on the treatment of total coliform bacteria by retention basins, and the observed treatment efficiency varied greatly. Davies and Bavor (2000) examined the treatment of enteric bacteria from a storm water retention basin, which averaged between -2.5 and 23 percent. García et al. (1997) found an algal pond and a waste stabilization pond to be greater than 98 percent effective at removing total coliform bacteria from domestic wastewater. García et al. (2008) also found an algal pond and a maturation pond to be greater than 98 and greater than 99 percent effective, respectively, at removing total coliform bacteria from domestic wastewater. Kurz (1998) studied the performance of shallow (1 m) and deep (2.75 m) retention basins for removing total coliform bacteria from storm water. The shallow pond demonstrated treatment efficiencies of 64 and 4.2 percent over a 5-day and 14-day retention time, respectively. The deep pond increased total coliform densities by 284.5 percent over a 5-day retention time, but decreased TC by 36.9 percent over a 14-day retention time.

A 2006 report by the USEPA (USEPA, 2006) reported on the treatment efficiency of four wet ponds for the treatment of bacteria in general (no specific species noted). One wet pond near Austin, TX removed 46 percent of bacteria, while a second nearby removed between 89 and 91 percent of bacteria, on average. A third pond in New York removed 86 percent of bacteria, and a fourth pond in Wisconsin removed 70 percent of measured bacteria densities.

Overall, the studies reviewed demonstrate that there is a high degree of variability in terms of bacteria treatment performance among retention basins. While some systems were able to attain high bacteria treatments, others increased bacteria densities during the monitoring period. The most influential and sensitive factors in treatment capacity seem to be proper design (including surface area, depth, and length-to-width ratio) and operation (i.e. hydraulic loading rate and residence time), as well as other factors such as land use characteristics (for storm water ponds) and additional sources of bacteria (i.e. waterfowl and other animals).

The studies reviewed pertaining to the effectiveness of retention basins for bacteria treatment are summarized in Table A-2.

**Table A-2: Summary of Studies Reviewed on Retention Basin Performance for Bacteria Treatment**

<table>
<tr><th rowspan="2">Reference</th><th rowspan="2">Wastewater Source</th><th rowspan="2">HRT (days)</th><th colspan="2">Influent Density (CFU/100 mL)</th><th colspan="4">Pollutant Treatment (%)</th></tr>
<tr><th>Fecal coliform</th><th>E. coli</th><th>Fecal coliform</th><th>E. coli</th><th>Fecal streptococci</th><th>Total coliform</th></tr>
<tr><td>Bavor et al., 2001</td><td>Storm water</td><td>Not provided</td><td></td><td></td><td>--</td><td>--</td><td>-2.5</td><td>--</td></tr>
<tr><td rowspan="2">Borden et al., 1998</td><td>Storm water (dairy farms/ woodlands)</td><td>20.2</td><td>120,100</td><td></td><td>48-90</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>Storm water (commercial, forest)</td><td>9.5</td><td>1,050</td><td></td><td>-5</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>Davies et al., 2000</td><td>Storm water</td><td>Not provided</td><td></td><td></td><td>--</td><td>--</td><td>--</td><td>-2.5-23</td></tr>
<tr><td rowspan="2">Garcia et al., 1997</td><td rowspan="2">Domestic (rural)</td><td>5</td><td rowspan="2">Not provided</td><td rowspan="2"></td><td>98.05</td><td>--</td><td>95.95</td><td>98.76</td></tr>
<tr><td>3</td><td>98.62</td><td>--</td><td>97.43</td><td>98.68</td></tr>
<tr><td rowspan="2">Garcia et al., 2008</td><td rowspan="2">Domestic (rural)</td><td>10</td><td rowspan="2">4.36-7.6 log CFU/100 mL</td><td rowspan="2"></td><td>>98</td><td>--</td><td>94</td><td>>98</td></tr>
<tr><td>20</td><td>>99</td><td>--</td><td>--</td><td>>99</td></tr>
<tr><td>Greuel et al., 2001</td><td>Storm water</td><td>Not provided</td><td>25,457</td><td></td><td>68</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>Hathaway et al., 2008</td><td>Storm water</td><td>Not provided</td><td>9,033</td><td></td><td>57</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>Hathaway et al., 2009</td><td>Storm water (residential)</td><td>Not provided</td><td>9,033</td><td>2,122</td><td>70</td><td>46</td><td>--</td><td>--</td></tr>
<tr><td rowspan="4">Kurz, 1998</td><td rowspan="4">Storm water</td><td>5</td><td>$2.29 \times 10^2$</td><td></td><td>98.2</td><td>--</td><td>--</td><td>64</td></tr>
<tr><td>14</td><td>$2.08 \times 10^3$</td><td></td><td>76.4</td><td>--</td><td>--</td><td>4.2</td></tr>
<tr><td>5</td><td>$1.59 \times 10^2$</td><td></td><td>88.5</td><td>--</td><td>--</td><td>-284.5</td></tr>
<tr><td>14</td><td>$1.57 \times 10^3$</td><td></td><td>69.2</td><td>--</td><td>--</td><td>37.9</td></tr>
<tr><td rowspan="3">Mallin et al., 2002</td><td rowspan="3">Storm water (residential, other)</td><td rowspan="3">Not provided</td><td>488</td><td></td><td>56</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>97</td><td></td><td>86</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>74</td><td></td><td>-15</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td rowspan="4">USEPA, 2006</td><td rowspan="4">Storm water</td><td rowspan="4">Not provided</td><td rowspan="4">Not provided</td><td rowspan="4">Not provided</td><td colspan="4">46</td></tr>
<tr><td colspan="4">86</td></tr>
<tr><td colspan="4">89-91</td></tr>
<tr><td colspan="4">70</td></tr>
</table>

## Summary

The results of this literature review suggest that constructed wetlands are a more reliable option for the treatment of bacteria in wastewater and storm water than retention basins, although a greater number of studies were available for constructed wetlands. Fecal coliform and total coliform bacteria are the most commonly reported constituents for both types of treatment systems, although considerable information exists for the treatment of *E. coli*, fecal streptococci, and other bacteria types (not included in this summary).

A review of studies on constructed wetland effectiveness suggests that constructed wetlands may be more reliable for the treatment of domestic and other wastewaters than for storm water, as treatment efficiencies were higher on average for the former. A significant degree of variability was observed among all four bacteria groups analyzed, suggesting that a wide range of factors are at play in determining overall treatment capacity. Further review and research are needed to better understand these factors. Overall bacteria treatment ranged from 0 to 100 percent, although the majority of studies reported bacteria treatment efficiencies in the range of 90 percent and greater. None of the studies reviewed reported an *increase* of bacteria densities in treatment wetlands.

Although relatively few studies were available on bacteria treatment by retention basins, the sampling of studies in this review suggests that their effectiveness for the treatment of bacteria is highly variable, ranging from a more than doubling of bacteria density (Kurz, 1998) to greater than 99 percent treatment of bacteria (García et al., 2008). Retention basins traditionally have been designed for the purpose of peak flow reduction and storm water management with less priority given to potential water quality benefits. Along with treatment wetlands, further review and research are needed to better understand the many factors that influence treatment performance, and to better design these systems for optimal water quality benefit.